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Recent Advances in the Treatment of Sewage Sludge.

The treatment and disposal of sewage sludge has been in many localities one of the most difficult and unsatisfactory problems with which the sanitary engineer has to do. It has been not only expensive for inland cities, but it has required very careful attention to guard against objectionable odors.

The septic tank has passed through an interesting cycle during the past ten years as regards its influence on the question of sewage sludge. Ten years ago much was said to indicate that the septic treatment would eliminate the sludge from sewage either entirely or nearly so. A few years later it was realized that the septic treatment would not cause a liquefaction of all suspended matter in the sewage, and many engineers expressed themselves as opposed to septic treatment for various reasons, the principal one of which seemed to be the difficulty of disposing of septic sludge without objectionable odors.

Quite recently there has been a marked advance in understanding the nature of the septic treatment, and the trend of opinion now seems to be toward a more intermediate and rational position than the extremes above mentioned. There has been, however, on the part of some engineers, a hesitancy in adopting the septic treatment on a large scale, because of the meagerness of practical results to confirm those theories which experienced experts consider most plausible.

For the past five years a notable influence upon several large sewage purification projects in this country, at least along indirect lines, has been exerted by the experiences at Birmingham, England, under the direction of Mr. John D. Watson. This metropolitan district now comprises a population of about 900,000 people, and has for several years been building works for disposing of its sewage by the septic treatment followed by sprinkling filters and their appurtenances. Perhaps in no way have Birmingham experiences been watched with more interest by sanitary engineers in this country than with reference to sludge disposal by the septic treatment. Elsewhere in this issue appears a valuable paper by Mr. Watson on sludge treatment, in which will be found the practical results of developments along this line at Birmingham.

A perusal of this paper will show that Mr. Watson is obtaining exceedingly satisfactory results in employing the septic treatment for separating suspended matter from the sewage and in disposing of the same on land. The results are notable in two ways; the sludge from the plant dealing with the sewage of 900,000 people is disposed of without any objectionable odors whatever; and this result is obtained at a very low cost, in round numbers about five cents per cubic yard of wet sludge.

In brief, Mr. Watson allows the sewage to flow through a series of septic tanks at an average lineal velocity of about 1.2 ft. per minute; the street wash and other coarse detritus are deposited in the first compartment of the septic tanks and removed about once a week; a large portion of what might be termed "grit chamber deposit" is pumped forward into the septic tanks; the deposit in the septic tanks is reduced to an inodorous humus mass by bacterial activity; and at intervals of about seven weeks or so, the inodorous septic sludge is pumped upon adjoining lands covering the same to a depth of some 8 to 10 in. It is particularly significant to note how definite is Mr. Watson's statement in his paper as to the inodorous and inoffensive character of the septic sludge obtained at the Birmingham plant. He points out that no objectionable odors are experienced as this sludge is applied to land. By absorption and evaporation the loss of water causes this sludge to be reduced to somewhat less than half its original bulk, and there results a fairly dry mass which thus far has been of little service for purposes of cultivation. Mr. Watson has plowed in the solid sludge with a steam plow. It appears, however, that this is an incidental feature and one upon which the sludge disposal is not dependent for its success. A field receiving sludge in this manner would, of course, when plowed, show for a few years a mixing of dry sludge with the soil, but gradually this mixing would become less and less and finally become eliminated as successive doses are applied to the same area.

It will be noted that the Birmingham procedure has been used as a basis for the disposal of sludge for the proposed new works for purifying the sewage of the City of Baltimore in accordance with the recent report of the Board of Advisory Engineers of that city. The report recently made upon the disposal of the sewage for the City of Paterson, New Jersey, indicates several methods for treating the sludge, the better one to be decided upon by actual experience. It is to be remembered, however, that the Paterson sewage would have its composition materially modified by trade wastes from textile industries. This question of the influence of the composition of the sewage upon the results obtained by the septic treatment is evidently one which must be carefully considered when arriving at the practical benefits of this treatment. In his paper Mr. Watson gives valuable average data for a period of two years, showing the composition of the Birmingham sewage.

In this country and in England some experiences with septic tanks have been in general harmony with those on a large scale at Birmingham. Other experiences, however, on both sides of the Atlantic have shown more or less variance, indicating the important differences either in the composition of the sewage or in both of these factors. It seems clear that the experiences at Birmingham ought to serve as a safe guide for quite a range of sewage problems elsewhere, although it is equally certain that if the influence of all local factors and conditions is ignored, that disappointments will result in some cases. Within certain limits the works must be built and operated to meet the individual character of the sewage and other local conditions.

At Mansfield, Ohio, experiences in the treatment of sewage sludge in septic tanks appear to coincide more closely with those of Mr. Watson at Birmingham than others in this country with which this journal is familiar. As stated by Mr. R. W. Pratt, chief engineer of the Ohio State Board of Health, in the August Proceedings of the American Society of Civil Engineers, the sewage at Mansfield averages about 1,000,000 gal. per day, and for about four years has flowed through a septic tank having a capacity of about 1,000,000 gal. The septic tank is about 50x100 ft. in plan and 8 ft. deep, indicating an average lineal velocity of about 1 ft. per minute. When this tank was emptied last spring, after it had been in continuous use for four years, it was found that the sludge, ranging in depth from 1 to 2 ft. over the bottom, had been reduced practically to an inodorous mass which could be readily air-dried upon land without creating offensive conditions.

In comparing experiences of Birmingham with those elsewhere, it now appears that the absence of objectionable odors is essentially due to the bacteria having consumed the crude or unstable portions of the organic matter which are deposited from the sewage as the latter flows through the septic tank. When these are reduced by the septic treatment to stable compounds, the likelihood of objectionable odors in the septic sludge is greatly reduced, if not entirely eliminated. In other words, the key-note to success seems to be, not to remove the sewage sludge from the septic tanks until the sludge has passed through the putrefying stage and actually become inert humus matter.

It is far more important to arrive at the result just stated than to strive for the liquefaction of a certain percentage of the suspended solids, or, as Mr. Watson described in his paper, the gasification of a certain percentage of the total solids. It is highly desirable to reduce the volume of sludge per million gallons of sewage, both by the compacting in the septic tanks (causing a reduced percentage of water in the sludge) and by the action of the bacteria. This reduction in volume of sludge means reduction in cost of disposal, but this item is of far less importance than the ability to secure freedom from objectionable odors.

The Utilization of Small Water Powers.

One of the curious features of the growth of electrical power transmission is the number of hydraulic powers that have been found in quite unexpected places and of which few people realized the existence. Very naturally most of the larger powers have now been at least discovered. There still remain, however, many falls on small streams which have escaped notice or have been considered too unimportant to survey. At the present time power transmission is in a condition which justifies the development of minor privileges, and engineers in knocking about the country ought to keep their eyes open for such possibilities. Engineers in the East have been so long accustomed to thinking of a water power as something with a big dam across an imposing stream, that the possibilities of a small stream under the high heads of a mountainous region may be overlooked by them. At, say, 20 ft. of head it takes a wheel a couple of feet in diameter and a flow of nearly 3,500 cu. ft. of water per minute to give 100 h.p. At 75 ft. head a 10-in. wheel will do the same work on about 850 cu. ft. per minute, and when several hundred feet is obtainable the water demanded is proportionately less, until at 650 ft. of head a mechanical horse power corresponds to a cubic foot per minute. A mere brook then suffices to give a power that may be worth at least developing for local use.

Such minor powers are common in any mountainous district, but the small flow diverts attention from them. Yet this very class has possibilities in the way of storage of water that are most attractive. It is indeed possible by creating a pond of very moderate size to store water enough to carry the plant over the entire dry season; or if a dozen or score of square miles of water shed are available, to create a useful power where only a scanty rivulet existed before. Prior to examining such a possibility, however, it is well to determine the minimum amount of continuous water power that is worth considering in connection with electric transmission, for the cases where these mountain powers can be utilized in situ are few and far between. The general cost of development and equipment tells part of the story, the cost of transmission the rest. In the hydraulic work the cost of conduits from the dam to the power house is generally the controlling item when considering high head, and this is again determined by the distance necessary to be covered. If the cost can be kept in the neighborhood of $100 per horsepower the outlook for an economical short transmission is good, since this means an annual charge of no more than $10 or $12 per horsepower for the motive power. The cost of wheels and generators with their equipment will commonly run from $25 to $30 per kilowatt in cases where raising transformers are not needed, the usual case for small powers. All of this can be approximated very readily, as also can the cost of the necessary buildings. The heaviest charges in small work come in the operating expenses and in the pole line. Pole lines for light wires need not cost more than $250 to $300 per mile, exclusive of wires and right of way.

The latter, in working on a small scale, is commonly along the highway, so that the cost is small; but the cost of wire, unless the line is short, may add considerably. Still, at a given voltage, the cost of copper per kilowatt transmitted is a constant, and the only relatively fixed item is the cost of stringing, which varies only slightly with the size of wire until the larger sizes are involved. For mechanical reasons, however, it is not desirable to string wire smaller than No. 4 or No. 5, so that the minimum cost of conductor is somewhere about $500 per mile. Fortunately, the depreciation charge against bare wire is practically negligible, and wire of this minimum size will carry comfortably the output of the class of plant considered.

The cost of attendance is the most serious output in small stations. It means generally the pay of at least three men, and occasional extras, not less than $2,000 per year even for a very small plant. At 100-kw. capacity this would come to at least $20 per kilowatt per year, which added to the other charges is pretty nearly prohibitive. At 200-kw. capacity the operating charge gets down to reasonable figures. In a rough estimate one will not go far wrong in saying that for electrical purposes a water power of 250 to 300 h.p. on steady flow is worth considering. Anything below this is of little account except for local utilization, and the usefulness of the power increases rapidly above this point. If the situation is favorable for storage, a good deal can be done with small streams, but unless the above amount can be made available without going to heavy expense, there is not much that can be done. If several such powers are available they can be often worked together to advantage. There are many powers near the limiting size that have been passed over as too small, and these are the ones which ought to be carefully looked after in the interest of small places and small industries.

The Water-Tightness of Concrete.

The degree of denseness, or approach to practical solidity attained in the fabrication of concrete is a measure of its imperviousness to the passage of water through it, and hence is of the greatest importance in many works where it is necessary that the masonry shall be as nearly water-tight as possible. With the wet mixtures of concrete now generally used in engineering work, concrete possesses far greater denseness and is correspondingly less porous than with the older, dryer mixture. At the same time it is exceedingly difficult to produce concrete in the large masses of actual work of such close texture as to prevent undesirable seepage at all points.

Many efforts have been made to secure the water-tightness of concrete in a practical manner, some resulting in comparatively tight material and others less satisfactory. It is not intended by these observations to state that practically water-tight concrete has not yet been attained under conditions where that quality is imperatively necessary, but rather to indicate the prevailing character of first-class concrete in this respect, even where it is made with scrupulous care. The success of many special developments and applications of concrete and reinforced-concrete construction depends to a material extent, and in many cases chiefly, upon the degree of approach to actual impermeability to water in those classes of material.

In general, there are two ways in which impermeability may be attained, one by making the concrete so dense or of such close texture in the first instance as to permit no water to seep through it, and the other by reducing the small seepage channels to such minute dimensions that the exceedingly fine material of almost infinitely small amount carried by the clearest waters may within a reasonable time choke the small seepage interstices, and so prevent absolutely all seepage, or by producing the same results by some other, but equivalent, procedure.

Obviously the only possibility of reaching the desired results by the first method is by reducing the voids in the combined mass of sand and gravel or broken stone to the greatest practicable extent, and then trusting to a finely ground cement to close those voids completely. Practical experience in the making of concrete has shown that this end is attained only by a careful balancing of the sand and gravel or broken stone, and by an equally careful determination of the minimum volume of finely ground cement required to fill the reduced voids. Even with these precautions and with a most thorough mixing, portions of the concrete may not be sufficiently dense to prevent all seepage. If, however, the cement be ground to sufficient fineness, and if it be in sufficient quantity somewhat more than to fill the voids a concrete practically impermeable to water will be produced.

One difficulty to be overcome is that which always arises with carefully balanced material, namely, the comparatively small volume of cement required to fill the relatively small quantity of voids necessitates an unusually extended mixing in order to distribute the cement completely and uniformly throughout the mass. If this complete and uniform distribution of the cement is not accomplished there will be parts of the mass with only partially filled voids offering paths of seepage. If these paths are of much extent the water will surely find them, and probably also find its way through the body of masonry in which they exist. These conditions explain why some parts of concrete hydraulic structures show seepage while others are actually water-tight, the latter having voids at least filled with finely ground cement, while the other parts are deficient in it.

It is obvious that if the unfilled portions of the voids offer but small seepage interstices, the particles of solid matter carried by the water may be too large to be carried through them, or if so small as to find their way to the interior of the concrete mass, the greatly decreased velocity of the seepage water will cause their deposition en route. In either case when the operation is sufficiently prolonged the seepage will gradually stop. The results of these operations have frequently been observed in connection with hydraulic structures of concrete. In the absence, however, of the conditions necessary to accomplish impermeability in this manner resort must be had to means effective to close paths of seepage too open to be choked with the finely suspended matter of the water. Various compositions of more or less excellence, to be proved by experience, have been proposed. A few of these are already of established character, while others must yet demonstrate their claim to engineering confidence.

A rather unique method of rendering porous concrete impermeable is exemplified by some recent experiments by Mr. Baldwin-Wiseman, a description of which will be found in another column of this issue of The Engineering Record. In these experiments water was forced through a rich concrete under pressure. While that pressure was constant, the rate of seepage of the water decreased with the time, showing a marked tendency to close the seepage passages. It is quite possible that the fine material carried in suspension by the water aided in producing this result. The experimenter, however, appears to be of the opinion that the water under pressure dissolves some of the material and then deposits it in stalactitic form near the exterior surface of the concrete, where the water escapes under much less pressure. There is no question that some of the calcareous material of the concrete is occasionally dissolved and then deposited in this manner, but it is doubtful whether reliance can generally be placed upon this procedure. It is so highly probable as to be practically certain that careful balancing of the materials, fine grinding of the cement, and the most thorough and wet mixing must be chiefly, if not wholly, depended upon to produce impermeable concrete.

The Plain Duty of the Hydraulic Engineer.

Altogether too frequently it happens that the practising hydraulic engineer is confronted with the uncomfortable fact that basic data relating to many features of his work are either altogether lacking or are existent in very meager and more or less intangible form. This unfortunate condition prevails to-day, not so much on account of a dearth of opportunity to acquire the information in question, as for other reasons. These appear to be, first, that many engineers having to do with works where data of extreme value can be procured are, or deem themselves to be, too busy to undertake their derivation, continuing to depend upon the work of others along these particular lines; secondly, that a large volume of important information, gathered at the expense of much toil, but under very satisfactory circumstances, is kept for private use alone. Happily the selfish spirit suggested by this last statement appears to be rapidly changing, and engineers generally, now-a-days, are far more apt to publish the facts gained privately from research, experiment and practice, than are the members of many another profession.

It is the purpose of this discussion both to call attention to the meagerness of fundamental data relating to various branches of hydraulics and of hydraulic engineering, and to advance a plea for the pursuit, in these directions, on existing plants, of more extensive studies whose results shall be published in a form and through media which shall be available to the profession at large. By way of suggestion certain fields in which research might profitably be conducted will be briefly mentioned, and in this connection it might be said that it is the constant aim and desire of The Engineering Record to place before its readers the results of dependable observations, which shall be of interest to any considerable body of men.

With the passage of time it is undoubtedly true that engineering studies and computations involve greater refinements and constantly increasing definition. This fact is the direct outcome of the accumulation of engineering data Perhaps in few other fields are these statements more true than of hydraulic engineering. For instance, the determination of friction losses in pipes under a variety of special conditions and the calculation of discharge through or over apparatus of particular design, are now capable of being made with a certainty almost undreamed of a few years since. Yet along these very lines the necessity for further information is most cogent. Filtration plants and power developments may be given as examples of engineering structures wherein are inherently embraced refined hydraulic problems. In the case of the first, the efficiency of design may depend in part upon the careful adjustment of friction losses and upon the measurement and control of effluent quantities; in the second, each detail may become of value, both scientific and commercial, measured by friction losses and the application of hydraulic principles on the one hand and by dollars and cents on the other. It is frequently stated that hydraulics can never become an exact science and that it is therefore unreasonable to expect to obtain or use values representing any high degree of refinement. This may be true where the several elements cannot be definitely known or assumed in advance, or where the conditions are imperfectly understood or are subject to important changes. It is believed, however, that there is a tendency to apply the argument too broadly, indicating the failure to appreciate the importance of accuracy where there is a legitimate call for it.

As indicating quite clearly the range of reliable results which are available for the determination of values with respect to that branch of hydraulic engineering which is connected with the flow of water in pipes and other channels, it is instructive to note the experiments to which reference is made by Messrs. Gardner C. Williams and Allen Hazen in their excellent book of hydraulic tables. A summary of the data to which attention is called in this book shows that a series of eighty-six experiments on closed conduits, made by thirty-nine independent experiments, and a series of seventy-eight experiments on open conduits, made by twelve independent experiments, were used as a basis of values and for comparison. The most striking feature brought out by a study of these data, the majority of which were published in detail in The Engineering Record for March 28, 1903, is the small number of available observations upon which the elucidation of important principles and the derivation of values for practical work has depended. While the authors of the book referred to would probably disclaim any attempt to exhaust the sources of information, it appears to be true that the most representative and authoritative experiments are generally included. In nearly every class of structure represented, there is normally occurring in practice a wide range of conditions upon which almost no satisfactory studies have been made. Many of the experiments quoted by the authors are the only ones whose results are as yet available for their respective types of structures, notwithstanding the fact that the studies were conducted many years ago and have been published in every work on hydraulics since written.

Referring to the experiments upon tuberculated cast-iron pipes, it is interesting to observe that in connection with a special study of the subject by Mr. F. C. Coffin, twenty-three series of observations were obtainable, performed by but twelve observers. In view of the fact that pipes more or less dirty and tuberculated exist in almost every water-works system in the world, the numbers of available friction values determined through carefully conducted experiments is surprisingly small.

The effect of different kinds of water and different classes of sewage on pipes and channels of varying sorts demands a more definite study and very much more specific information than exists at present. Very few data exist with respect to the action of filtered waters or those heavily charged with alkali, upon cast-iron and other pipes. Moreover, it does not appear that the hydraulics of waters heavily charged with salts or loaded with sediment are as yet satisfactorily demonstrated.

Although wooden-stave pipe has come rapidly into use in the East as well as in the West, the number of experiments on these structures is likewise very small, and the results of the several observations made up to the present time have not been thoroughly in accord. It is evident that much work in this field is desired at once, especially with respect to the effects of different kinds of waters during various periods of time.

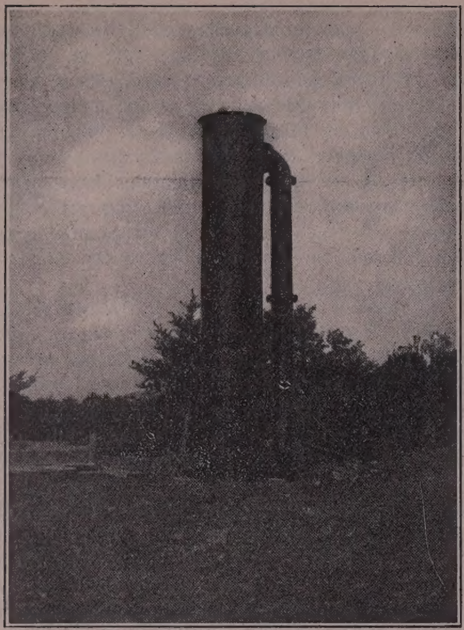
Another fruitful field for extensive studies is in connection with large artificial channels employed in power development, irrigation and other plants. Among the most notable investigations in this direction are probably those which have been pursued for many years at Lawrence and at Lowell, Mass., by some of the best hydraulic engineers that this country has produced. The results of these investigations have never been published, but the engineering profession awaits with keen interest the dissemination of this information, which will have the greatest merit and value.

The splendid work of the hydraulic laboratory of Cornell University in demonstrating on a practical scale coefficients for various crests, such as appear on large dams, marks an important step. It is probable that the majority of engineers are not in touch with the results obtained thus far, although fortunately certain features of the work have recently appeared in print. In connection with the determination of the coefficients for flow over dams actually in place there is an excellent chance to perform a service. The United States Geological Survey is doing something in this line in connection with its most important and valuable work of investigating the regimen of rivers in various parts of the United States.

Important as is the entire subject of water hammer in connection with the designs of distribution and power-line systems, the available definite, practical information is pitifully meager, and there is abundant opportunity for the conduct of observations on many features of this problem.

No effort has been made herein to do more than suggest a very few lines along which, in connection with existing hydraulic works, experiments and observations of broad scope may profitably be undertaken. In the realm of theoretical hydraulics there are many features of great importance which require accurate demonstration, but which are frequently beyond the province of the practising engineer. It must be evident that the field of research is a broad one and that almost every hydraulic engineer has the opportunity to conduct trustworthy investigations whose results should have a very real value for the profession at large.

Subaqueous Rock Cutting is being carried on quite successfully in increasing the ruling depth of the Manchester Ship Canal from 26 to 28 ft. The rock is pulverized by blows from a series of hardened steel hammers mounted on a barge, and raised and lowered by hoisting engines. The hammers carry circular cutters forged from a single ingot of armor-piercing steel and have a removable ogival point similar in form to that of large projectiles. These points may be replaced without necessitating the renewal of the cutter bars, which weigh 10 tons each. The bottom of the canal has a width of 120 ft., and consists of sandstone of varying degrees of hardness. The cutters are caused to fall vertically on the same spot until a sufficient number of blows has been given to pulverize the rock to a depth of 3.5 ft. The number of blows given depends on the hardness of the material, about 2 cu. ft. being pulverized per blow in average material. The spots on which the cutters are dropped are generally spaced about 3 ft. apart on centers, although this spacing is varied as the character of the rock changes. The pulverized rock is removed with a bucket dredge. The results of the process as stated by Mr. W. Henry Hunter, chief engineer of the Manchester Ship Canal, in Engineering, show that the average rate of advance over the full 120-ft. bottom width of the canal is 36 ft. a day; 6,403 cu. yd. of rock have been broken up on the average each month at a total cost of less than 18 cents per cubic yard.



Jacketed Steel Standpipe.

The Pipe Line of the New Gravity Water Supply of Lynchburg, Va.

A new municipal water supply, of 8,000,000-gal. daily capacity, for the city of Lynchburg, Va., is to be derived from water impounded by a dam across Pedlar River, about 21 miles from the city, and about 110 ft. higher than the Lynchburg high service reservoir, as described in The Engineering Record for July 23 and 30, 1904, and May 12, 1906. The pipe line, 30 in. in interior diameter and about 110,000 ft. long, consists of approximately 99,000 ft. of wood stave pipe, 7,000 ft. of lock-bar steel pipe and 4,000 ft. of cast-iron pipe, the latter being used under the crossings of the James River, and other streams, and in the streets of Lynchburg. The wood pipe is used for pressures up to 83 lb. and the steel pipe for higher pressures. The alignment is such that the wood pipe will be under pressure at all points, and under all conditons of service, thus insuring constant saturation and a maximum durability of the material. The hydraulic slope is one in one thousand and the location is such that nearly all of the pipe is laid in trench averaging about 6 ft. in depth. It also passes through three rock tunnels aggregating about 2,000 feet in length, with a 6x7-ft. section, the greater part of the tunnel work being lined with concrete. The minimum radius for the wood stave pipe is about 200 ft. Curves in cast-iron and steel pipes are made by specials.

The steel pipe is all ¼ in. or ⅜ in. in thickness, of the lock-bar type, made by the East Jersey Pipe Co., Paterson, N. J. It is made from sheets 30 ft. long, which are placed singly in a special upsetting machine to which one full-length edge is clamped by hydraulic jacks. Both edges are successively planed and upset by a traveling head, and the plates are removed, curved and assembled with the lock bars. The jaws of the lock bars are then closed under hydraulic pressure, and the pipe sections are thus made with the longitudinal seams secured by continuous smooth bars which are believed to offer considerably less frictional resistance to the flow of water than a riveted pipe. It is shipped in 30-ft. lengths which were joined in the trench by cast-steel sleeves, the joints being poured with lead and caulked like hub and spigot joints. The maximum head on the steel pipe is about 400 ft. It is the first main of this type manufactured.

The steel pipe was coated with Pioneer mineral rubber coating at the mills. This compound makes a very good protective coating to prevent corrosion; it is, however, so soft that it is easily scraped off. After the steel pipes were received at the trench, therefore, they were placed on skids and carefully inspected. All marred spots were marked with chalk and the paint gang, by applying a plumber's lamp, would melt the original coating, which was from 1/16 to ⅛ in. thick, and flow it back over the damaged surface coating. It was necessary to scrape off the coating at the ends of the pipe before the lead joints could be poured. After the joints were calked the finished pipe was then inspected, inside and outside, and all abraded surfaces painted with a heavy coat of thick Smith's durable metal coating. The 30-in. cast-iron pipe is 1 in. thick, except for the James River crossing, for which the thickness was increased to 1.2 in., is of standard dimensions, and was very carefully tested and inspected.

The maximum pressures on the pipe line are limited by 12-in. cast-iron overflow stand pipes at four summits. In cases where the topography is such that the hydraulic gradient is at considerable height above the adjacent grounds the standpipes are vertical, located on an offset from the main, in a 3-ft. riveted-steel shell seated on a concrete foundation. The space between the standpipe and the shell is packed with coke screenings and the top of the shell is roofed. The upper ends of stand pipes of this class, which are about 40 ft. high, are surmounted by return bends passing through the protecting shell and continuing in a vertical length of pipe reaching down parallel with the stand pipe to below the surface of the ground and into the concrete foundation, where a 90-deg. bend discharges into a vitrified pipe drain debouching into an open gutter. In the other types where the topography permits, the stand pipe is carried under-ground up the hill side, and terminates at the upper end in a special bend discharging into a small concrete covered manhole, and thence to an open gutter.

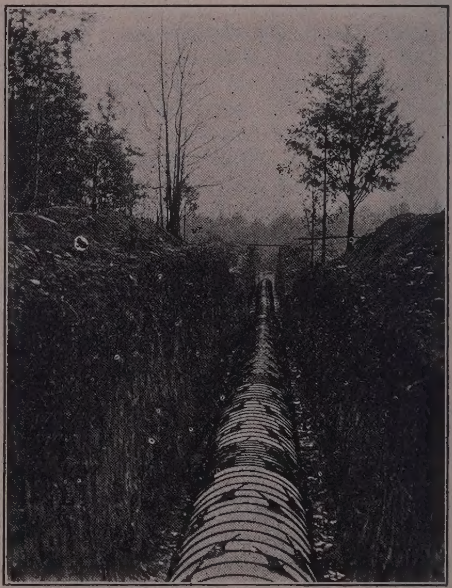
Every summit in the line is provided with an air valve. The 2 and 6-in. valves, of which there are about 90 in all, were furnished by the Glamorgan Iron Works. In several cases the 6-in. air valves are arranged in clusters of 2 or 3, and of automatic construction operating to admit air should the water be suddenly allowed to escape from the pipe line, as would happen in case of a rupture. These valves are not intended to allow air to escape while the pipe is in service. They are proportioned to admit air enough if a break in the pipe should occur below them, to prevent the unbalanced external pressure from exceeding 8 lb. per square inch. The 2-in. air valves are of the float-and-lever type, and will allow air entrained in the water to escape while the pipe is under pressure. These are placed on intermediate low summits. Several small semi-units, however, are equipped with ¾-in. Loetzer air valves, or valves which must be operated by hand.

Between Lynchburg and the James River crossing, nine miles from the city, there are two deep valleys, the hills on either side of which are too low to permit the construction of stand pipes. At these points the pressure is relieved by automatic spring relief valves.

In all of the principal valleys there are 8-in. cast-iron blow-offs and at the James River crossing, where there is a head of 400 ft., the blow-off is provided with double valves.

The cast-iron pipe is carried in a trench over the embankments surrounding the circular Lynchburg reservoir in an inverted siphon, and at the center of the reservoir connects directly with the high service supply main and with a standpipe 60 ft. high. This standpipe passes through an ornamental octagonal concrete pier, above which it is enclosed in a riveted-steel shell 6 ft. in diameter, which extends a little above the top of the stand pipe and is finished with a proper cornice. The water overflows the top of the interior pipe and descending between it and the outer shell emerges through perforations in the bottom of the latter, below the top of the concrete pier, in an ornamental basin surrounding the shell, over the edge of which it flows on all sides, and forms an attractive fountain about 15 ft. above the surface of the reservoir. This arrangement provides aeration and also insures constant pressure for the high service district, the excess not consumed in the high service district overflowing to the large reservoir for the general supply of the city. The fountain and standpipe are made attractive features of the reservoir.

The wood-pipe staves are cut from 2x6-in. redwood planks, from 10 to 24 ft. long, and are shipped from California in box cars ready for assembling in the pipe. They are all of first quality clear stuff with inner and outer surfaces planed to the proper curves to fit the circumference of the pipe. One radial edge of each stave has an ⅛-in. head 1/16-in. high, which under pressure of the band cinching is forced into the side of the adjacent stave. A kerf is sawed into the end of each stave to receive a 2x⅛-in. flat-steel joint plate ¼-in. longer than the width of the stave, to cut off all leakage at the butt joint. The pipe is banded with ½-in. round homogeneous mild steel rods having an ultimate strength of 58,000 to 65,000 lb. per square inch. Each band has a hemispherical head on one end and a cold rolled thread on the other. The bands are proportioned so that they will break in the body of the bar, rather than under the head or in the thread, as all tests have demonstrated. The bands are spaced on the pipe in accordance with the formula $N = 3.3 H.$, applicable only to 30-in. pipe built under the specifications adopted, where N is the number of bands per 100 ft. and H is the presure head in feet. For heads of less than 50 ft. the bands are spaced as follows: From zero to 20, 108 bands per 100 ft.; 21 to 26, 114 bands; 27 to 31, 120 bands; 32 to 36, 127 bands; 37 to 40, 134 bands; 41 to 45, 150 bands; 46 to 50, 165 bands. There are altogether about 320,000 bands for the 2,000,000 ft. board measure of staves.



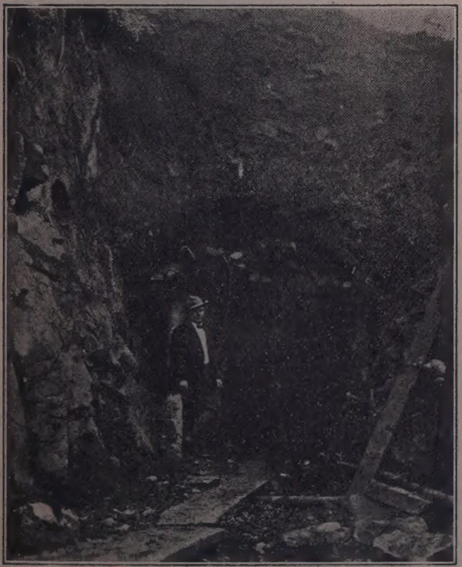
A Length of Wood-Stave Pipe.

The bands engage special malleable cast-iron shoes differing somewhat from previous patterns and designed especially for this work. They are accurately calculated and the frictional area of the base proportioned so that they will slip around on the pipe during the cinching of the band rather than cut into the wood. The angle of the back face is such that the nut will tend slightly to draw against the pipe, rather than away from it, and a shoulder is provided to engage the forged head so that it cannot become displaced. The metal in the shoes is not less than ⅛-in. thick in any portion and they are strong enough to break the bands. The shoes are so well annealed that they can be flattened on an anvil without breaking. They were designed to weigh one pound each and the actual average weight is 97 lb. per 100. About 160 tons of shoes are used on the work.

On the wood stave pipe a gang of 12 men to fit up and assemble, and six more men for the back cinching, averaged about 150 ft. of completed pipe for a day's work. Four such gangs were generally employed and where the adjacent sections met, the joints were buckled in by cutting the staves ⅛-in. long and springing them to place.

As the pipe was assembled in the trench, and the bands approximately spaced, the foreman measured and marked the location of each band on the pipe in accordance with the spacing table with which he was provided. The nuts were then run up with a brace wrench until they commenced to tighten, after which the back cinching gang finished the adjustment with S wrenches, having handles not over 10-in. long, as a precaution against over-cinching. As the pipe staves were received sometimes many months before they were used the ends were likely to become split and checked notwithstanding the care taken in piling them and protecting them from the weather. The contractor was, however, allowed to cut off the injured portions and use the remainder provided a length of 10 ft. or more of perfect timber could be secured. Notwithstanding the utmost care in cutting them it is found almost impossible to saw them accurately enough by hand to make perfect joints; with hand cut staves the joints are likely to be open inside or outside.

The most scrupulous pains were taken to thoroughly protect all steel and iron from corrosion. The pipe bands were shipped straight, and after bending to the proper circle were tied up in bundles of about 25 and dipped in a tank containing hot Pioneer mineral rubber pipe coating, where they were allowed to remain until they acquired the temperature of the coating mixture. After being hauled to the trench they were again inspected, and wherever the coating was abraded by handling or transportation were retouched with Smith's durable metal coating before being applied to the pipe. During the cinching the hammering on the band injured the original coating, and all such spots were again painted before the trench was beck-filled. The cast shoes were also protected by being dipped in the same coating before shipment, and repointed wherever scars were noticed after assembling.



Entrance to One of the Tunnels.

All bends in the wood stave pipe have a radius of 200 ft. or more, are made by springing the loosely banded pipe and bracing it to the required curve before tightening the bands, driving the butt joints back tightly during the cinching. Shorter radius curves are made with cast-iron specials.

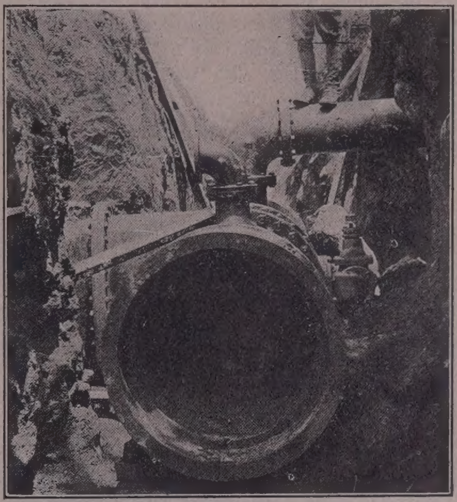
The junctions between the wood pipe and the cast-iron pipe are made by inserting the former in the special 7-in. bells of the latter, pouring the joints with lead and caulking as for cast-iron pipes. The junctions of wood and steel pipes are made with a special bell and flanged casting leaded to the wood pipe and bolted to the steel pipe. Steel and cast-iron pipes are joined with bolted flanges and rubber gaskets.

The 30-in. gate valves are set in the wooden pipe with special bell and flange castings which have top outlets for connecting the 6-in. air valves. Standpipe connections are made to the wooden pipe with 12x30-in. cast-iron tees. Valves are connected to the steel and cast-iron pipe with flanged joints, except for the smaller sizes, which are poured and caulked. All steel pipes are provided with a manhole at convenient points to facilitate future painting if necessary. In some cases the steel pipe is laid on very steep grades, where, if a man should slip in the pipe he would be unable to climb to the top; exit manholes are provided at the bottom of these grades. Where the pipe is carried down steep slopes it is thoroughly tied in by concrete anchors to prevent sliding. On side-hill crossings of small brooks, and similar places, the pipe is usually protected by a concrete wall built parallel to it on the lower side and carefully back-filled so as to force the water to flow over the top of the pipe and thus avoid danger of undermining.

A Section of Wet Trench.

The pipe was handled from the railroad at three points and was distributed along the line by teams hauling it at an average of nine miles, over extremely bad roads. The trenches are about 6 ft. wide, the upper part of which was generally excavated, where practicable, with teams and scrapers, which were also used to handle the back fill.

Most of the pipe laying was comparatively easy but difficulty was encountered at the Pedlar River crossings. For these crossings the stream was either diverted around, or carried across the line in flumes, the trench was sheeted and the water was kept out as well as possible with centrifugal pumps. The pervious character of the gravel in the river bottom, however, was conducive to very wet trenches, and usually only single lengths of the cast-iron pipes could be laid at one time. The excavation in some of these places was through a few feet of boulders and gravel with a comparatively shallow rock cut, and was of course very difficult. At the James River crossing there was fortunately a long island in the center of the river on the line of the pipe and it was possible to build a dam diverting the flow from one channel so that the pipe could be laid there in open trench, after which this dam was removed and one built across the other channel enabling the pipe to be laid there in the same way.



Connection of Valves to Pipe Line.

During the earlier part of the construction two or three accidents occurred to the wood pipe on account of delay in back filling trenches in low sections. In consequence of this neglect heavy rain storms filled the trenches and floated the pipe, raising it in some instances 5 or 6 ft. The raised portions of the pipe, and those adjacent to it at the bottoms of the hills, were very carefully inspected, and the pipe taken out between the extreme points where there was a visible opening of the butt-joints; it was then rebuilt at the original grade, using the old staves when not damaged and replacing those damaged with new ones.

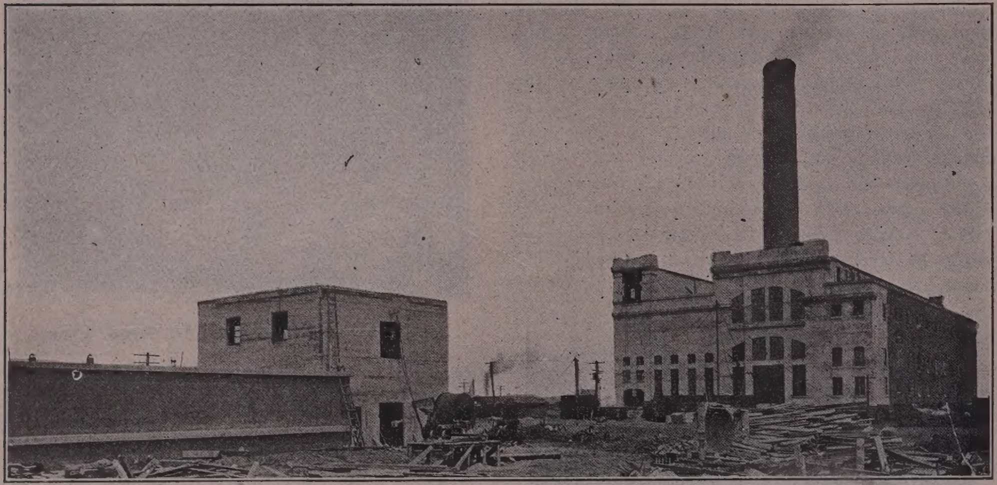
The city has established a telephone line following the pipe line to the Pedlar River dam, with stations at convenient points accessible to the city's employees in charge of the work. Some difficulty has been experienced with the line during severe electric storms, to which this region is subject. On one occasion during a thunder storm the line was struck near tunnel No. 2, the charge traveling along the line for two miles, and leaving it at a telephone station, to the injury of a number of persons standing nearby.

The approximate cost of the pipe line is about $400,000. All but two miles of the wood pipe are now laid, together with about ⅔ of the cast-iron pipe and practically all the steel pipe.

The works are being built in conformity with the report and recommendations of Mr. James H. Fuertes, of New York, who has been consulting engineer to the water committee of Lynchburg city council since he indicated to that city how it could get a new water supply for an amount within the financial reach of the community. The works are being executed under the direction of Chief Engineer H. L. Shaner, city engineer of Lynchburg. Mr. P. B. Winfree, now resident engineer at Pedlar River dam, was formerly resident engineer of the Pipe Line. He was succeeded, when transferred to the dam, by Mr. Paul Hooker, the present resident engineer. Mr. Chauncy G. Williams, of New York, is the general contractor. The sub-contract for manufacturing the staves and assembling the wooden pipe was given by Mr. Williams to the Excelsior Wood Stave Pipe Co., of San Francisco and New York, but the trenching and back filling was done by the general contractor, who also laid the steel and cast-iron pipe.

The Westport Power Plant at Baltimore, Md.

An electric generating station with a capacity of 33,000 h.-p. has recently beeen completed and placed in operation at Baltimore, Md., by the Consolidated Gas, Electric Light & Power Co. The station has a capacity at least equal to the maximum demands for power that have thus far been made on the company, and was designed and built so that additions to it could be made without interfering with the operation of the existing equipment. The station building is built entirely of reinforced concrete and is believed to be one of the largest, if not the largest structure, of that type of construction that has been erected. The station is advantageously located near the edge of the city on the Patapsco River, where a site embracing 23 acres of land was secured, lying between the tidewater line of the Western Maryland R. R. and the Curtis Bay branch of the Baltimore & Ohio R. R. Its close proximity to these railroads and with water connection to the Canton branch of the Pennsylvania R. R. gives it the advantage of transportation facilities from the three great coal-carrying roads entering Baltimore.



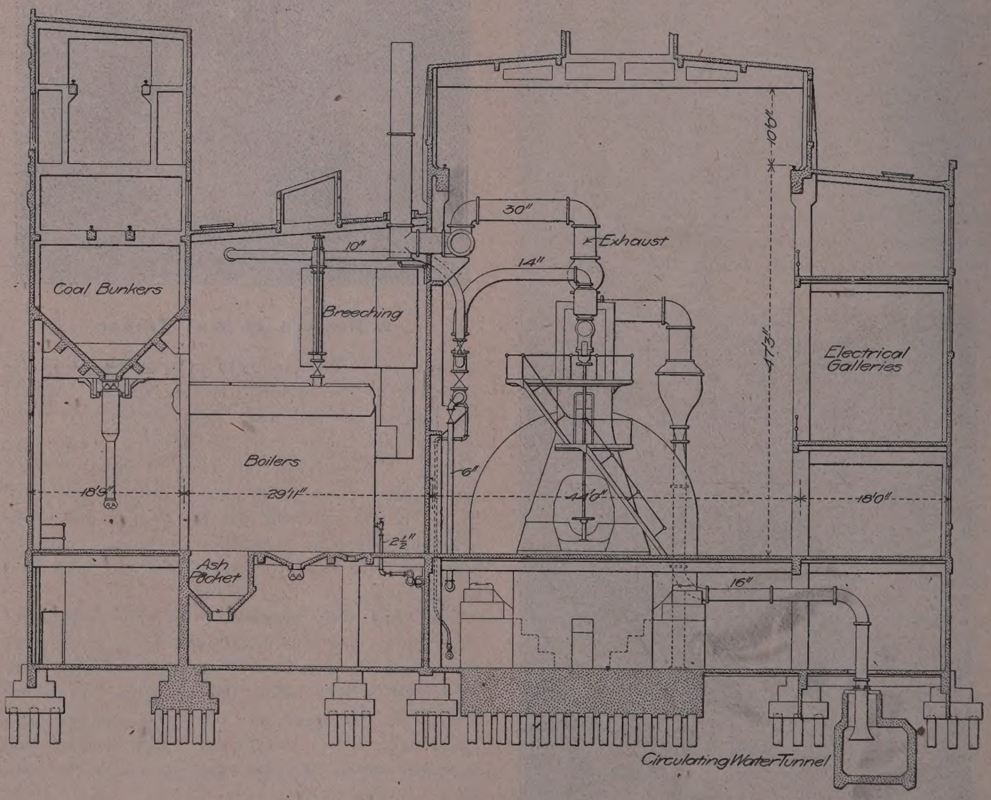
The New Westport Power Station at Baltimore, Md.

The company's experience during the recent conflagration in Baltimore determined the location of the new station at Westport for the following reason: 1. Isolation from adjacent buildings, eliminating exterior fire hazard. 2. Transportation facilities, which include all three coal-carrying railroad systems entering Baltimore with additional delivery by water from the Baltimore & Ohio R. R. station at Locust Point. 3. Storage capacity sufficient for a coal supply for a whole year. Coal strikes, railroad strikes, blockades and congestions or any other cause that might temporarily halt the supply of coal from the mines can therefore have no effect on the plant. 4. Room for extension as found in the large area which the company has acquired, giving ample space for expansion of the plant to supply the needs of Baltimore, however great they may become in the future. 5. Water supply for the boilers and for condensing purposes in abundance.

Ground was broken for the new station on Aug. 16, 1905, and the first engine was turning on June 15 of this year, or ten months after. As above stated, the station is built entirely of reinforced concrete, to the entire exclusion of wood—foundations, walls, floors, pillars, galleries, coal bunkers, and discharge chutes for ashes from the furnaces all being built of concrete. The framing for windows and doors is of steel. In the center of the building is the largest smoke stack in the vicinity of Baltimore. This stack is built of radial brick, has an inside diameter at the top of 14 ft. and is 209 ft. high. In addition to natural draft furnished by this chimney the boiler room is equipped with forced draft to provide for any sudden demand for power.

The power house structure is 110x255 ft. in plan, of simple, yet pleasing, architectural treatment of the exterior. The building is divided into a boiler and an engine room, which are 48 and 62 ft. in width, respectively. The engine room has a clear inside height of 60 ft. in the central portion and is surmounted by a monitor for ample lighting of the main machinery floor and the operating galleries at the outer side. The boiler room is but 40 ft. in height excepting at the outer side where the coal bunkers are located, and above the bunkers is the runway for a coal-handling system. The entire structure is planned with special reference to ease of extension when future growth of the system will make increased demands on the station.

The steam generating equipment of the plant consists of eight Babcock & Wilcox water-tube boilers, each of 650-h.-p. capacity, which are arranged in single setting in a row, with the firing floor at the outer side of the room. The building accommodates ten units, but two have not as yet been installed, inasmuch as the entire generating equipment provided for is not all in place. The boilers are designed for operation at 150-lb. pressure and have superheaters. The furnaces are fitted with plain grates, and are hand-fired, bituminous coal being used. The products of combustion are carried to the stack by ample breeching connections at the rear, which lead into the stack on either side, as shown in the accompanying plan drawing. The boilers are fed by an equipment of three large Worthington duplex feed pumps, the feed water being preheated by three open feed heaters of the star-vacuum type built by Warren Webster & Co., Camden, N. J. The feed system consists of a 7-in. header extending the length of the boiler room with 3-in. branches to each boiler.



Cross Section of Power House.

The coal is stored in the spacious bunkers above the boilers and is thence fed directly by gravity to the boiler room floor. These bunkers are filled by coal handling machinery of the latest type so that the coal in transit from the barge or cars is automatically handled throughout. The boilers discharge their ashes through reinforced-concrete chutes into cars which run on an industrial track in the basement. The ashes are then run out under a gantry crane, and are handled by the latter, being used eventually to reclaim waste land.

The gantry crane extends from the wharf to the power house and is 293 ft. long. It is driven by a 50-h.-p. motor operated through an electric controller designed by the Morgan Engineering Co. Under normal conditions the bucket trolley passes out of the building to the gantry structure and thence, without moving the gantry, direct to the wharf where the bucket is loaded, hoisted and again returned to the building. Should it be desired, however, to take coal from the coal piled in storage in the yard the bucket trolley passes out of the building on the gantry which is then moved over the coal pile where the bucket is loaded; the crane is then moved back opposite the opening into the building and the bucket trolley passes into the building as usual, depositing its load in the hopper of the crusher trolley. The structural part of this gantry was provided by the Lauer & Harper Co., of Baltimore, although all the driving mechanism and other machinery in connection with it was furnished by the Morgan Engineering Co., of Alliance, Ohio.

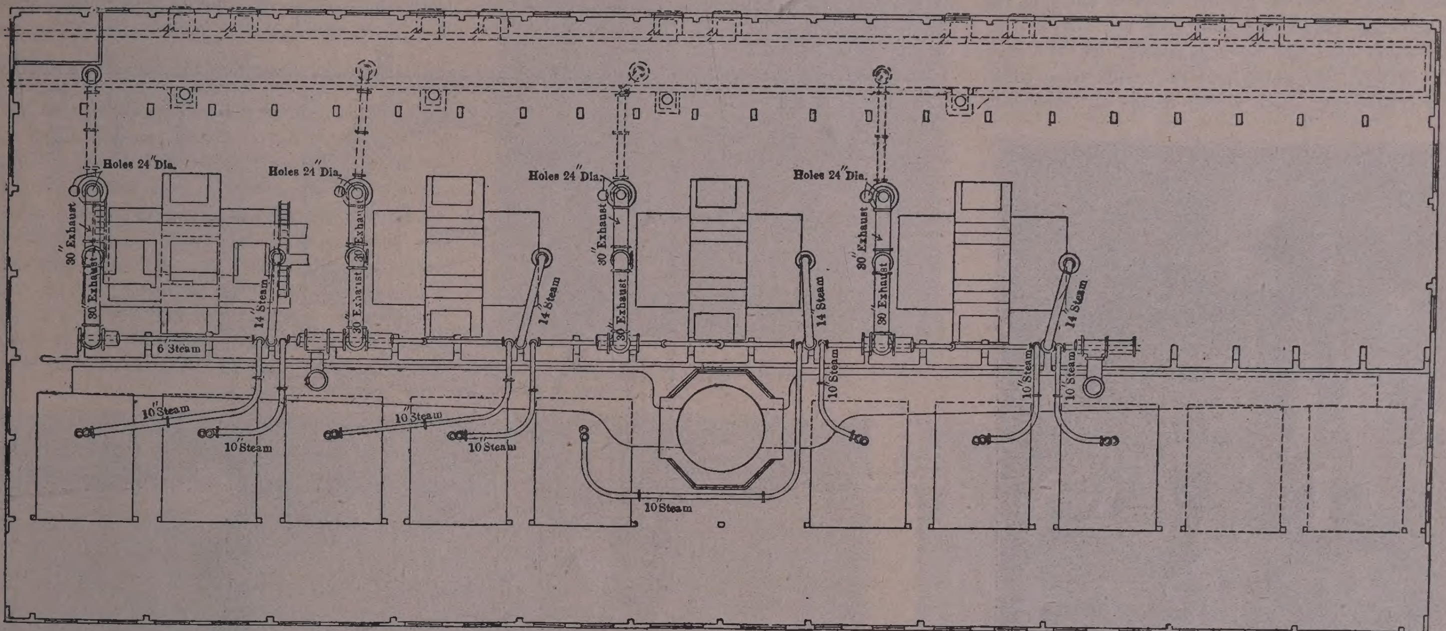
The bucket trolley carries and operates a large clam-shell bucket, especially designed for handling coal. This trolley is supplied with three independent electric motors which in turn are operated by controllers placed in a cage attached to the trolley. The main hoist motor is of 50-h.-p. capacity, the bucket-operating motor 25-h.-p. and the trolley-travel motor 25-h.-p. The opening and closing of the bucket is accomplished by a very ingenious arrangement of differential gearing which gives the operator perfect control of the bucket at all times and enables him to open and close it entirely independent of the hoisting and lowering mechanism of the bucket.

For this class of work, in order to economize time as much as possible, it is desirable that the bucket should lower very rapidly from its highest to its lowest position. To accomplish this, the main hoist motor has a compound winding and the lowering of the bucket is controlled to a nicety by a 60-point rheostat with armature short-circuiting resistance by means of which the bucket can be dropped very rapidly and stopped at any point quickly and easily by the operator in the cage. As stated before, the operator's cage is attached to this trolley so that the various motions are under direct control at all times, whether working outside the building on the gantry crane or inside the building over the coal pockets.

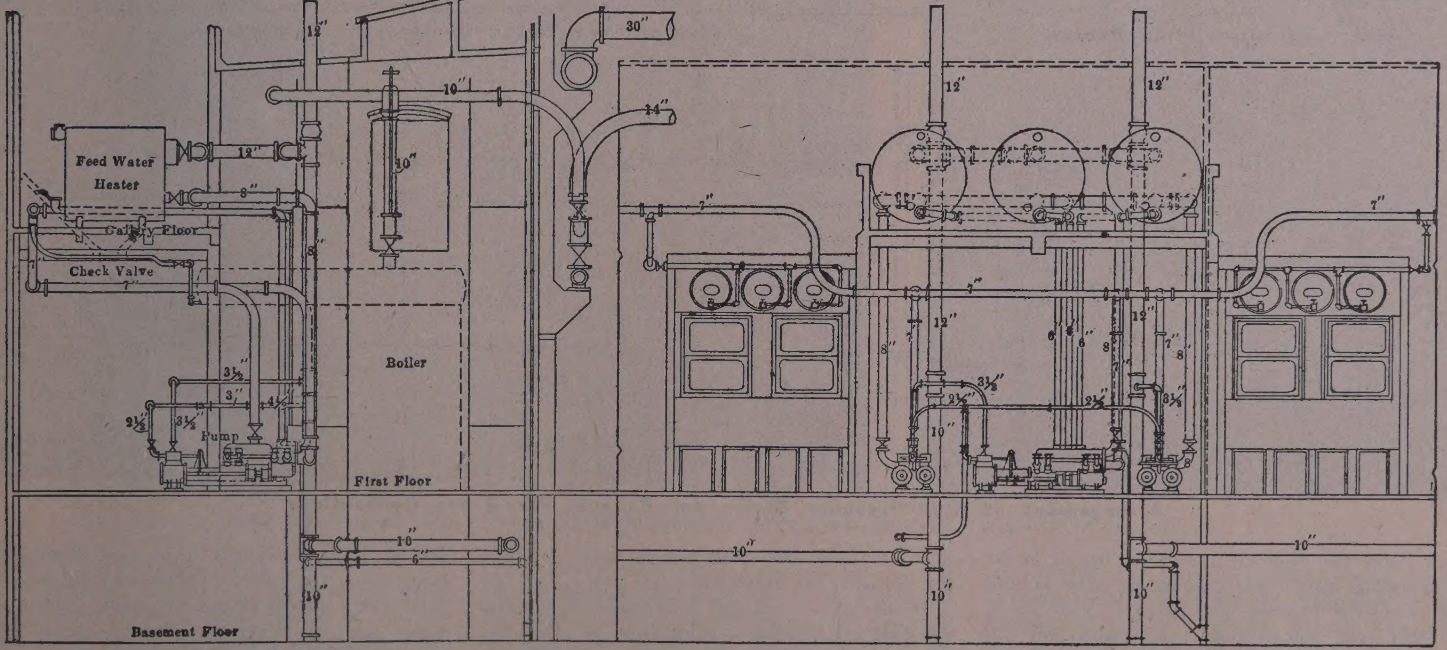
In the building over the coal pockets there is provided an electrically-operated crusher trolley or carriage built by the Morgan Engineering Co. This trolley is equipped with a hopper which is of structural steel and which is made in such a manner as to be easily filled by the bucket trolley operating over it. At the lower end of the hopper is the crushing mechanism driven by a 10-h.-p. motor. The cross travel of the trolley is operated by a 5-h.-p. motor on the operator's platform. The crusher trolley can be moved over any one of the several pockets in the building and the coal crushed and emptied into the desired pocket. This trolley is at all times under the control of the operator located on a stationary platform in the building.

The necessity, on machinery of this kind, to cover all the gears, motors, and other apparatus to prevent the dust and coal from getting into them has been carefully considered and taken care of by the builders of the coal-handling machinery, and in all parts provision has been made for the constant lubrication of journals and bearings and care taken to reduce friction to a minimum. The motors are all of the direct-current type operating at a potential of 220 volts. The approximate hoisting speed is 100 ft. per minute and the approximate cross travel speed is 650 ft. per minute.

From the crusher the coal passes to the coal pocket, thence through a cut-off valve to a one-ton traveling hopper which delivers coal to the boiler room floor. There are at present four hoppers in use. Poke holes are provided to loosen up the coal should it have a tendency to clog. The coal-handling machinery is capable of delivering coal from the barge to the field at the rate of 150 tons per hour and from the barge to the bunkers at the rate of 60 tons per hour.

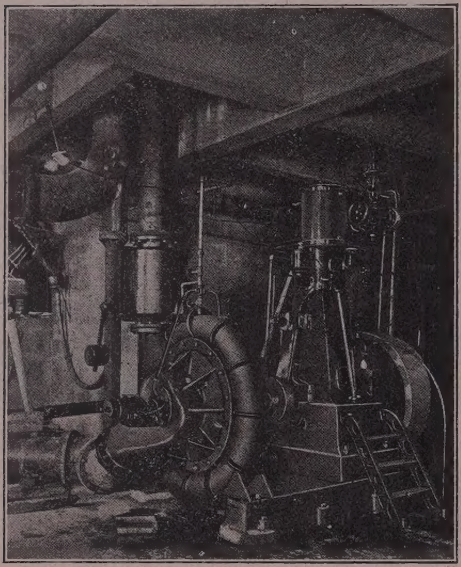


Plan of Power House, Showing Arrangement of Apparatus Installed.

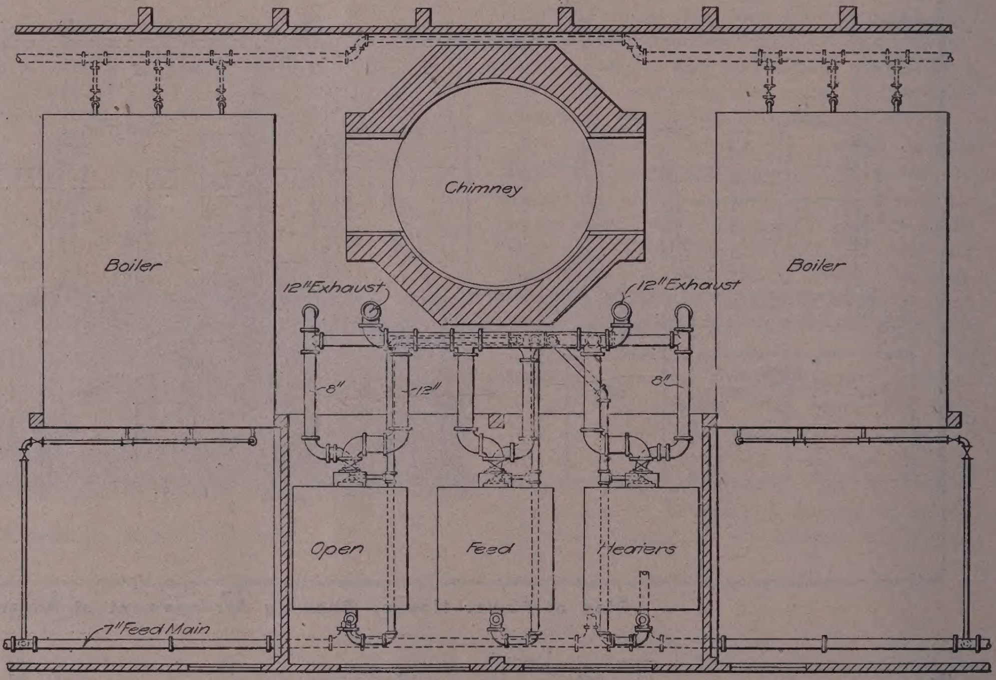


Elevation of Piping Connections between Feed Pumps, Heaters and Boilers.

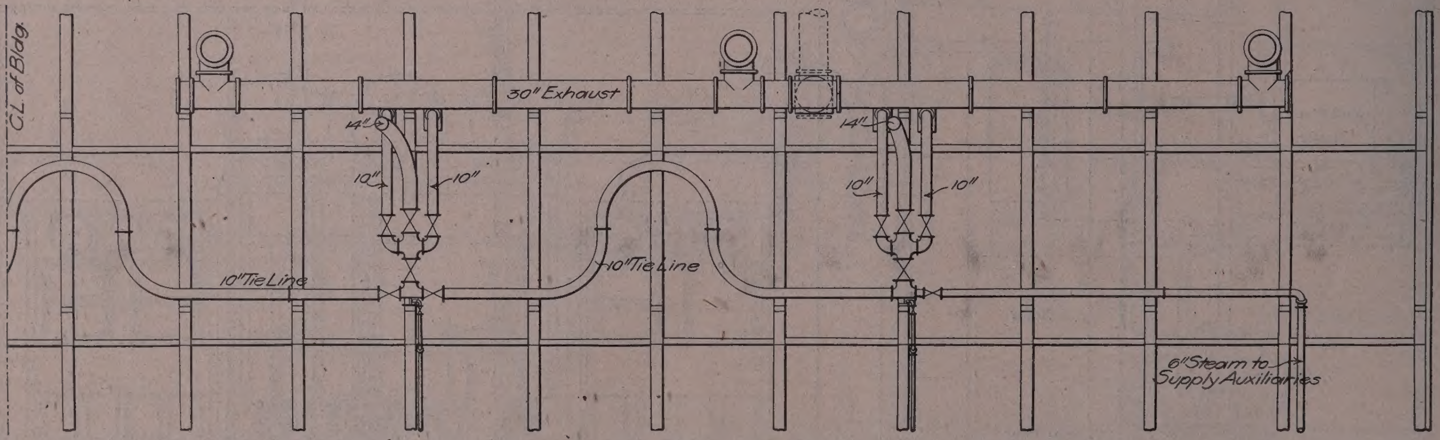
An ample, though not very clean supply of feed water for the boilers is obtained from Gwynn's Falls in a small stream near the station, from which it is pumped by a small pumping plant under the Baltimore & Ohio R. R. bridge passing over the falls to a purifying plant for the removal of sediment. After being filtered it is stored in a reservoir constructed for this purpose holding 1,750,000 gal., and situated between the station and the filtering plant. The water is pumped by two motor-driven, 5-in. vertical submerged centrifugal pumps, through 8-in. bell and spigot pipe to the coagulating basin about 1,200 ft. distant. Before entering the coagulating basin alum and lime are mixed with the water and act as coagulants. In the pipe line between the pump and the coagulating basin is a controlling valve which works automatically and keeps the water at a uniform height in the coagulating basin. The coagulating basin is built of reinforced concrete and is 40 ft. wide by 50 ft. long, and has a capacity of 200,000 gal., allowing thus three hours and ten minutes sedimentation in the tank before it reaches the filters. The coagulating basin is provided with two blow-off valves so that the sediment which collects in the basin may be blown off at intervals.



Centrifugal Circulating Pumps.



Plan of Feed Water Heaters and Connections.



Arrangement of High-Pressure Steam and Exhaust Piping and Connections.

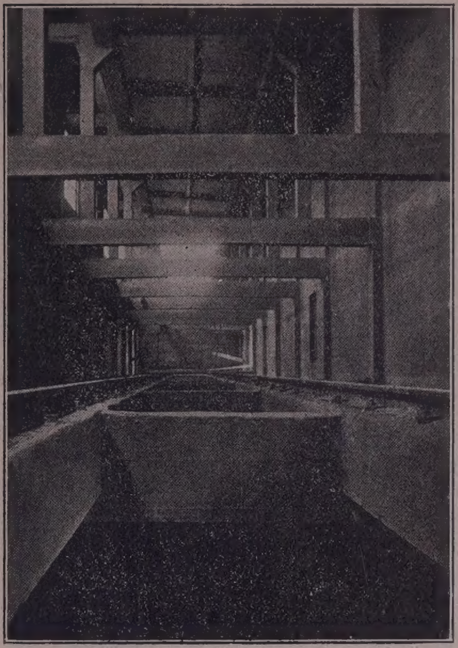
Three mechanical filter beds exactly similar to those built for purifying public water supplies are provided. These filters each have a sand bed area of 160 sq. ft. and a capacity of 500,000 gal. per 24 hrs. The underdrainage system of each bed consists of a longitudinal cast-iron manifold pipe at the center of the bottom into which galvanized laterals are screwed on 11-in. centers. Strainers screwed into these laterals hold back the sand and gravel of the filter bed and admit the filtered water. On top of the strainers is placed a 9-in. layer of gravel and on top of this gravel is placed a 30-in. layer of specially prepared sand of a uniformity of about 0.40 mm. with a coefficient of about 1.5.

Running lengthwise of the bed, where the coagulated water enters the filter bed, is a specially constructed wrought-iron box through which the water flows into the bed. After the water passes through the sand it is collected in this main collector pipe which runs to the outside of the filter bed, and on the outside of the filter bed is a valve discharging into a rate controller located in a canal running in front of the beds. This rate controller is of such construction that the flow of water from the bed is fairly uniform, the variation not exceeding 5 per cent. The rate controller can be changed to increase or diminish the flow. Connection is made from the discharge pipe of the filter beds next the rate controller to a loss-of-head tube. This is about 6 in. in diameter and is fitted with a float. On top of the tube is a gauge which registers at all times the loss of head or friction in the sand caused by the water passing through it. There is also an electric alarm connected to this tube which causes a bell to ring when loss of head is sufficient to indicate that the filters need washing.

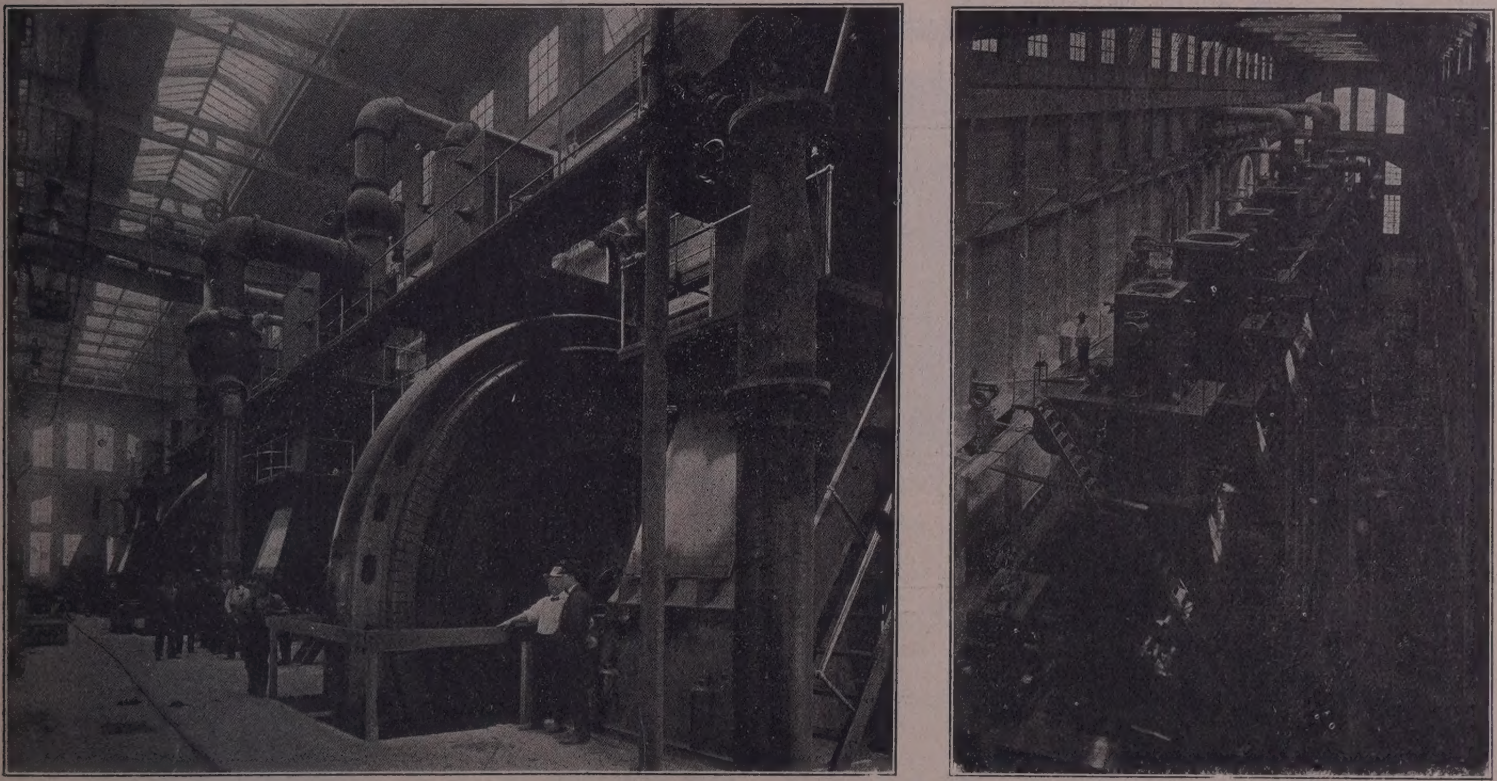
The discharge pipe from the filters is also connected to an 8-in. pipe which is connected to a tank in a tower located outside the filter house for washing the filters. On top of the gravel layers and underneath the sand layers is an air pipe system consisting of brass pipes placed 4 in. apart, by means of which air is forced so as to thoroughly wash the bed in connection with water. When a bed needs washing, the coagulated water is shut off through the pipe leading to the bed, and the discharge from the filter is also shut off. The valve connecting to the wash line is then opened and at the same time air is turned on in connection with the water. The dirt collected on top of the filters is then washed over into the wash overflow box which is connected with the sewer. The air is used in connection with the water for about three minutes, after which it is shut off and the water used alone for about one minute and a half longer. From the canal a pipe leads to the clear water storage basin previously mentioned, from which connections are made to pumps for supplying the local water system around the plant. Coagulated water may be run direct to this clear water basin should anything happen to the filters, and raw water can also be run either direct or through the filters without going through the coagulating basin should this be desired.

The filter house and filters are built of reinforced concrete, there being over the filters an office and also a room where the coagulants are mixed, two tanks being used for the alum and two for the lime. The discharge from these tanks empties into a regulating box which has an adjustable orifice. From the regulating box the coagulant is discharged into the raw water pipe and thence delivered to the coagulating basin. The pipe connections are also arranged so that the coagulant can be put into the water after it has passed part way through the coagulating basin, if it is found at times that the water does not require so long a period of coagulation. The filters are designed to remove practically all the turbidity and color from the water and at least 97 per cent. of the bacteria. There are no traces of the coagulant in the filtered water. The object of using the lime is to neutralize the alum coagulant if the alkalinity of the water is running low. From the reservoir, the water is pumped into the heaters and a 70,000 gal. tank placed 65 ft. above mean low tide for gravity distribution. The filtering system was installed by the Norwood Engineering Co., of Florence, Mass.

The steam piping systems are well illustrated in the accompanying detail drawings of the plant. The high-pressure system involves independent groups of piping between adjacent boiler and engine units, rather than the cross-connecting header system more generally in use, the various independent units being here tied together for emergency use by 10-in. interconnecting lines, which do not, however, serve the purpose of a receiver header. The arrangement of apparatus is such that each engine is supplied with steam by the two boiler units located opposite. These boilers deliver through 10-in. connections, each with non-return stop valves, to a 10x14-in. cross fitting, which connects on its upper side into the 14-in. engine supply connection and on the lower side to the cross-tie lines by which the various groups of units are tied together. As shown in the accompanying piping elevation, the arrangement of valves is such that any group of two boilers may supply the adjacent engine entirely independent of the remainder of the plant, during which the interconnecting tie line may be in service through the T-fitting below the cross; if either boiler of the group is out of service, it may be cut out and replaced by an auxiliary supply through the tie line from either side. The cross fittings and the tie line are drained by a high-pressure drip system, as is also the engine supply branch which consists of a 14-in. long-radius bend with a Webster separator at the throttle valve.



View in Coal Bunkers.



Views in Engine Room Showing Arrangement of Condensers and Exhaust Connections.

The atmospheric exhaust connections for the various engines consist of two 30-in. pipe headers carried on the division wall above the high-pressure steam tie line and fittings, with branches of similar size to each engine and near its middle a riser connection to an exhaust head above the roof. The engines are operated condensing, so that little duty will be brought on this system. The back-pressure valves are located in the engine branches immediately upon the exhaust outlet connections. There is also an auxiliary exhaust system consisting of two 10-in. lines in the boiler-room basement, one in either end of the plant, into which the auxiliary steam using machinery exhausts. These lines deliver into the feed water heaters which are located upon an elevated platform in the boiler room in front of the base of the stack.

The engine equipment of the plant consists of four vertical compound condensing engines having a rated capacity of 3,000 h.p., with 50 per cent. overload, or thus a total maximum capacity of 18,000 h.p. The fifth engine to be installed will be a combined horizontal and vertical compound condensing engine coupled to a 5,000-kw. fly-wheel alternator. This engine will have cylinder dimensions of 38 and 78 in. by 56 in., and will develop 7,500 h.p. with a maximum capacity of 11,250 h.p., making the aggregate capacity of the plant 30,000 h.p. These engines are built by McIntosh, Seymour & Co., of Auburn, N. Y., and embody the usual construction of the engine built by that company, with the exception of the larger engine which is being built by license under the patent granted the Allis-Chalmers Co.; this engine will be similar to the engines in the power house of the Interborough Rapid Transit Co., in New York City.

The condensing apparatus consists of an individual Worthington barometric jet condenser for each engine, of special design to meet the particular requirements of service. These units are of the well-known elevated type consisting of a cone, in the interior of which is located an air cooler. This cooler performs the very important function of assisting in the removal of all non-condensible vapors from the cone, which circulate through a series of passages around which injection water is constantly circulated at its lowest temperature, insuring that these vapors will be reduced to the smallest volume and eliminated from all suspended water. At the bottom of each cone is a special throat piece covered by patents of Mr. P. O. Keilholtz, which automatically takes the air vapors from the air cooler and introduces them into the falling column of water in the barometric leg. The injection water to each condenser is supplied by a 12-in. Worthington special pattern, volute centrifugal pump driven by a Reeves high-speed engine. Rotative dry vacuum pumps are used in connection with these units only at the time of starting the condensers. Their action is to assist in removing from the system all non-condensible vapors in order that the centrifugal pump may work under normal operating conditions.

The water for condensing purposes is taken from the Patapsco River through a double reinforced-concrete tunnel, having the intake below and the discharge above. Each of these passages is 6 ft. high by 8 ft. wide. Ears are provided on the side into which the suction pipes project. The top of the tunnel is even with the water at mean low tide, while the bottom is 15 ft. below mean low water. A weir is placed in the discharge tunnel so that the upper tunnel is always kept full of water; by this means the ends of the discharge pipes are never uncovered and the vacuum lost. Screens are provided in the intake tunnel to keep out all foreign matter, which are of No. 8 brass wire with 2-in. mesh enclosed in an angle iron frame sliding in grooves provided in the concrete. Two of these screens are placed in series, so that one may be cleaned whenever this is desirable without rendering it possible for foreign matter to find its way to the suction.

The electric generating equipment consists of four 2,000-kw., 25-cycle, 13,000-volt, three-phase General Electric alternators, direct-connected to the prime movers, and one 5,000-kw. machine, to be installed later. There are no transformers in the station and current at a potential of 13,000 volts is sent through oil switches to the other stations operated by the company. The switch cell and bus-bar structure is built of reinforced concrete on the second gallery, the twenty switches used being of the Westinghouse type C make. Directly above the switches and separated from each other by concrete partitions are the bus-bars. These are in duplicate and in sections, the sections being connected by knife switches, between the oil switches and bus-bars and also between the generator leads and oil switches, so that access may be had to any oil switch by simply pulling the knife switch. Motor-generators are used to supply the exciting current and also to charge 54 cells of battery which float on the exciter system. The battery is situated in one corner of the top gallery, the rest of which is utilized for men's lockers. The second gallery is devoted entirely to high-tension apparatus, such as switch cells, and bus-bar structures. It is worthy of note that all of the auxiliaries in the station are steam-driven and some of them operate against a back pressure of 25 pounds, their exhaust being delivered into the receivers of the large engines so that they operate practically as condensing engines.

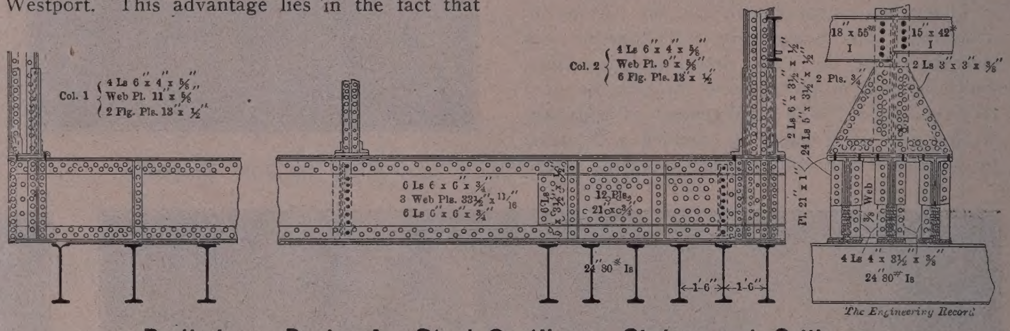
From the Westport station electricity is transmitted in lead-covered cables placed in subways built by the company from the power house to a junction with the municipal subway, which carries cables to the Pratt and Penn Sts. station, the McClellan St. station and the Monument and Constitution Sts. station, the Pratt St. power house of the company supplying the entire street railway system of Baltimore. The Pratt and Penn Sts. and the Monument and Constitution Sts. stations are now steam driven; but after the energy from Westport is received, the steam engines will be shut down, disposing of whatever objection there may be to the smoke from these stations. The McClellan St. station and the other two stations are equipped to receive the energy from Westport. This equipment includes the machinery for transforming and reducing the voltage and current.

Important as is the immunity from fire hazards and guaranty of a non-interrupted coal supply to the customers of the company, there is still another advantage obtained in the plant at Westport. This advantage lies in the fact that 25-cycle electric apparatus has been installed which has the same characteristics as the equipment of the Pratt St. power house of the United Railways. As these stations are tied together it is possible to interchange electricity between the two in case of accident.

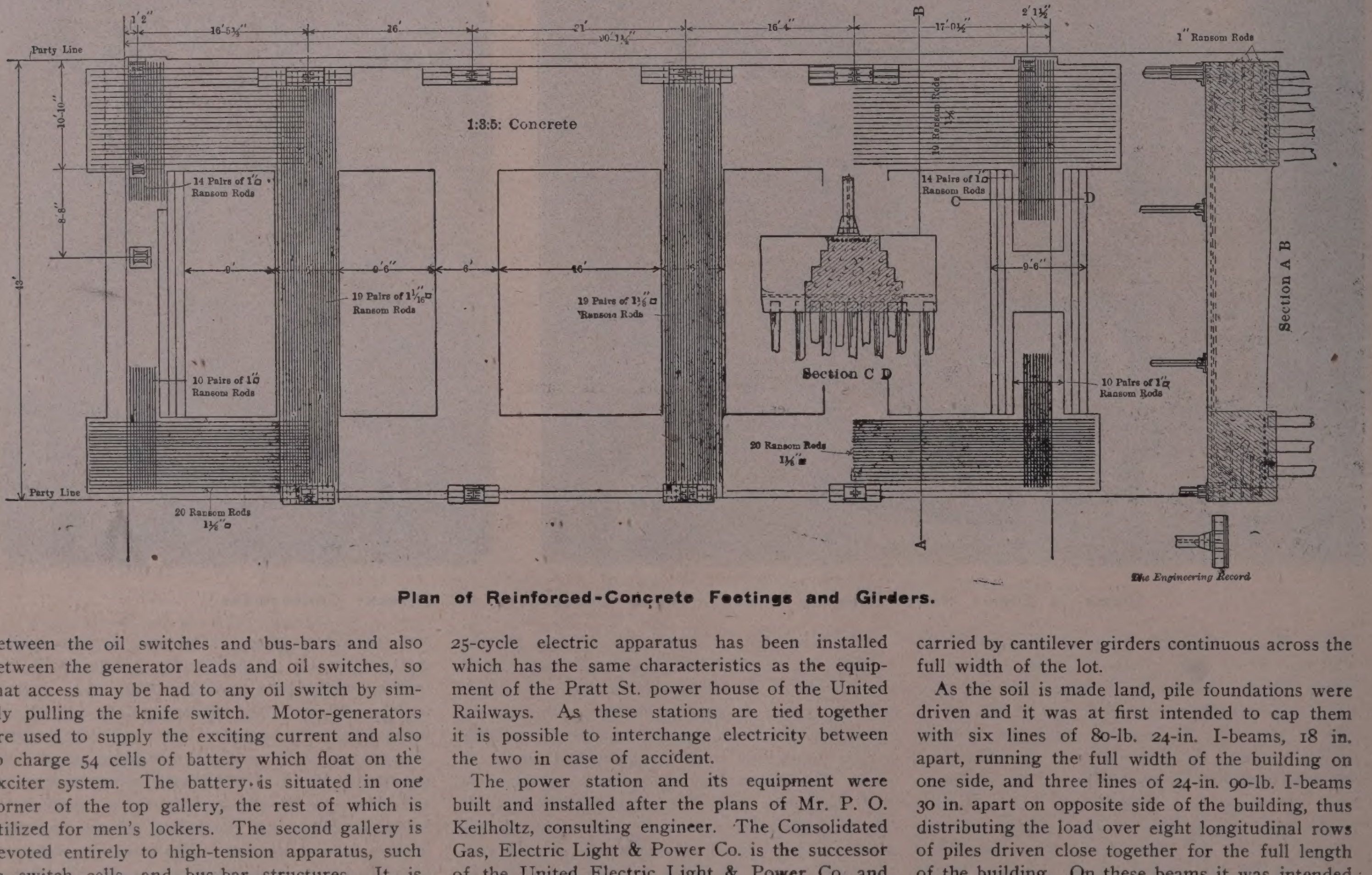
The power station and its equipment were built and installed after the plans of Mr. P. O. Keilholtz, consulting engineer. The Consolidated Gas, Electric Light & Power Co. is the successor of the United Electric Light & Power Co. and owns a large majority of the stock of the Consolidated Gas Co. Mr. Alten S. Miller is vice-president of the company, Mr. P. O. Keilholtz is the consulting engineer, Dr. Douglass Burnett is the manager of the electrical department, and Mr. W. Stuart Symington is secretary and treasurer. Mr. S. Davies Warfield is chairman of the board of directors of the company and of the executive committee of the board.

Comparative Designs of Steel and Reinforced Concrete for Cantilever Foundation Girders.

The 43 x 39-ft. addition to the Boston Athletic Association's Building, Blagden St., Boston, has a steel framework, with transverse floor girders spanning the full width of the building and supported by columns at the ends and in four intermediate pairs set as close as possible to the party walls on both long sides of the building. The architects, Messrs. Sturgis & Barton, estimated that the necessary thickness of side walls could be secured more cheaply by supporting new 16-in. brick walls on the steel framework at each story than by underpinning the adjacent old walls and increasing their thickness to the required amount. In order to avoid disturbing the old foundations and to secure symmetrical loading on the new ones, the columns were designed to be carried by cantilever girders continuous across the full width of the lot.


Col. 1 4 Ls 6" x 4" x 5/8", Web Pl. 11" x 5/8", 2 Flg. Pls. 13" x 1/2"
Col. 2 4 Ls 6" x 4" x 5/8", Web Pl. 9" x 5/8", 6 Flg. Pls. 13" x 1/2"
6 Ls 6" x 6" x 3/4"
3 Web Pls. 33 1/2" x 11/16"
6 Ls 6" x 6" x 3/4"
24" 80# Is
1'-6"
18" x 55# I
15" x 42# I
2 Ls 3" x 3" x 3/8"
2 Pls. 3/4"
4 Ls 4" x 3 1/2" x 3/8"
24" 80# Is
The Engineering Record

Preliminary Design for Steel Cantilever, Girders and Grillage.



Plan of Reinforced-Concrete Footings and Girders.

As the soil is made land, pile foundations were driven and it was at first intended to cap them with six lines of 80-lb. 24-in. I-beams, 18 in. apart, running the full width of the building on one side, and three lines of 24-in. 90-lb. I-beams 30 in. apart on opposite side of the building, thus distributing the load over eight longitudinal rows of piles driven close together for the full length of the building. On these beams it was intended to seat four transverse cantilevers each carrying a column at both extremities and proportioned to distribute the loads equally over the longitudinal I-beams. The cantilevers were designed with triple plate girders, 33½ in. deep, having heavy reinforced webs and flanges as indicated in the accompanying detail drawing. The columns were designed with extended bases wide enough to extend across the full width of the three girders forming each cantilever, and made integral with the column shaft so as to serve as pedestals and distributing girders.

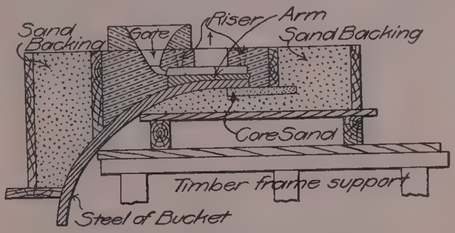
It was intended to protect the girders and I-beams by enclosing them in a solid mass of concrete with a minimum thickness of 6 in. outside the steel. The amount of concrete required for this purpose was so great that it appeared extravagant to employ it thus without utilizing its strength, so a comparative estimate was made for the design in which the concrete could be reinforced and made to form an integral part of the girder itself. The calculations showing that a considerable economy was thus attained, the second design was adopted and the steel cantilevers and I-beam grillages were superseded by cantilevers and distributing girders, or footings of reinforced concrete. The distribution girders, capping the piles are made of the same exterior dimensions as the concrete footings of the first design, thus effecting a practical saving of the considerable difference in cost between the I-beams and the reinforcement rods. The cantilever girders were of much larger cross-sectional dimensions than were those designed for the steel-plate girders.

The foundation exclusive of the engine foundations contains about 480 piles, 1:3:5 Atlas Portland cement concrete and Ransome bars and has been built without necessitating any shoring or underpinning of the adjacent buildings. The concrete footings enclosing the pile tops form a continuous mass from 3 to 9 ft. wide around the four sides of the lot, crossed by the four intermediate transverse cantilevers 6 ft. wide. At the ends of the building the footing is stepped off to a narrow width on top where it receives the brick walls. On the sides it has a rectanglar cross-section, with the upper surface in the plane of the tops of the transverse girders. At the ends of the building the columns are not supported on cantilevers, but have their loads distributed both transversely and longitudinally over the concrete footing by single tiers of twisted steel rods 1 and 1⅛ in. square arranged in successive horizontal tiers as indicated in the accompanying drawings. The four intermediate cantilever girders are 6 ft. wide and from 4½ to 5½ ft. deep reinforced with twisted rods arranged close together in single tiers near the upper surface as shown by the drawings.

The 18-hour Special Train of the Pennsylvania R. R. between New York and Chicago made a remarkable record during the first year of its operation, ending June 11, last. During the 365 days of service this train arrived late at Jersey City but 54 times and at Chicago only 37 times. The equipment caused delay 17 times during the year to the eastbound, and 14 times to the westbound train; the remainder of the delays were due to various causes.

Welding Broken Dredge Bucket Arms in the Field.

The broken bucket arms of clam-shell dredges of large capacity now being used in the construction of a harbor in the eastern end of San Pedro Bay, 27 miles from Los Angeles, Cal., on the Pacific Ocean, have recently been welded by the Thermit process near the site of the dredging operations and the buckets placed in service again with very slight loss of time. The harbor construction in which the dredges are employed is being carried on for the Los Angeles Dock & Terminal Co. by the North American Dredging Co. The national government is also making extensive improvements at the outer end of San Pedro Bay, which was originally an exposed roadstead with a shallow channel leading up to the point where the terminal company is now carrying on its improvements. The latter are entirely independent of the government work, however, and will be reached by an artificial channel having 30 ft. of water at mean low tide and protected by breakwaters.



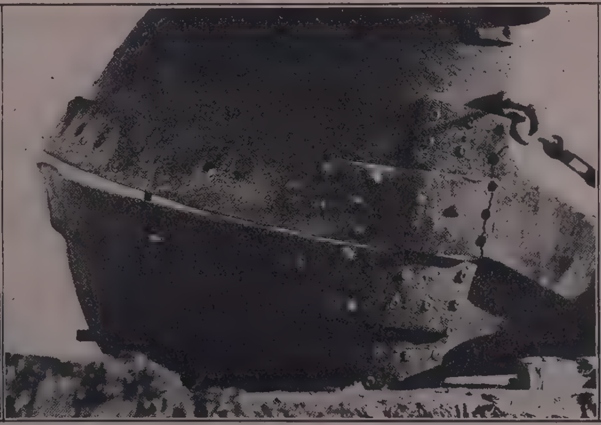
Arrangement of Mold.

The buckets of the dredges used in this work have a capacity of from 4½ to 6 cu. yd., those of the latter size weighing 12 tons without any attachments, when empty. The welding of this broken arm on one of the 6-yd. buckets is indicative of the manner in which the repairs were carried on. The breaks on this bucket occurred both on the outside and inside arms, at the junction of the latter with the bucket and through the first line of rivet holes. Immediately after the break the arm separated completely from the bucket, the space between the two being 13 in. This space was closed up by heating the opposite arm, after which it was kept closed by a chain and turnbuckle. The bucket was then picked up an laid on its side on a scow on which the welding was done.

The Thermit welding process utilizes the chemical reaction between finely divided aluminum and iron oxide, producing liquid steel at a temmperature of about 5,400° Fahr., without heat or power from the outside. The reaction between the powdered forms of aluminum and iron oxide is started by applying a match to a chemical ignition powder. During this reaction the iron oxide is broken down, its oxygen combining with the aluminum to a slag, while the iron being the heavier sinks to the bottom of the crucible in which the reaction is allowed to take place.

In welding these bucket arms a light staging was built up around the bucket to facilitate operations, the bucket having first been firmly blocked in position. A mould consisting of a mixture of fire clay and sand, made up in two parts in iron flasks, was then placed around the fracture. Owing to the extremely high temperatures existing during the process of welding this mould had to be made absolutely dry to prevent the generation of steam and a resulting porosity of the weld. When the mould had been placed it was well luted and banked up with dirt on all sides to prevent the possibility of leakage. The finely powdered chemicals were then placed in a conical-shaped magnesia-lined crucible suspended from the staging directly over a pouring hole in the mould. Immediately before filling the crucible the parts to be welded were heated to a red heat with a blow torch. The crucible was then filled and the chemical reaction started. This reaction lasts about 30 seconds, but a few seconds more than that were allowed in order to permit the steel and slag to become thoroughly separated in the crucible. The steel was then tapped into the mold by driving in a small pin suspended from the bottom of the crucible, the outlet of the latter being directly over the gate of the mold. The steel flowed into the mold, through the fracture and around the parts to be welded, and on cooling amalgamated with the latter and formed a collar or reinforcement which is shown in one of the accompanying illustrations.

Bucket Set for Repairs.



Fracture on Outside Bucket Arm.

Repaired Bucket in Operation.

The weld on this arm required 250 lb. of the mixture of aluminum and iron oxide, to which were added 56 lb. of steel rivets. The steel produced by the reaction between the aluminum and iron oxide contains 0.10 per cent. carbon, so the steel rivets were added to increase the tensile strength of the resulting steel. When larger welds are made by this process it is customary to add 15 to 20 per cent. of steel punchings, which are completely reduced during the reaction.

The repairs were carried on under the supervision of Mr. L. Heyenmann, consulting engineer for the Goldshmidt Thermit Co.

A Mines Draining Scheme has been projected for the vast lignite deposits to the northeast of Marseilles, France, which involves a 12.5-mile tunnel leading to the sea. The deposits are considerably above sea level, and the soil in their vicinity is so saturated with water that nearly 21,000 gal. of water have to be pumped for every ton of fuel that is mined. The tunnel will discharge water 59 ft. above sea level, and can be utilized by a railway on which the lignite may be transported to the port of Marseilles. It is said no serious difficulties are expected to be encountered in driving the tunnel.

The Puddling Effect of Water Flowing Through Concrete.

In the summer of 1904 Mr. W. R. Baldwin-Wiseman conducted a series of experiments on the rate of flow of water through a specimen of concrete, identical in composition with that used in the construction of the new graving-dock at Southampton. The experiments extended over a period of 50 days, readings being taken daily or at more frequent intervals as occasion demanded, and an account of them has recently been published by the Institution of Civil Engineers.

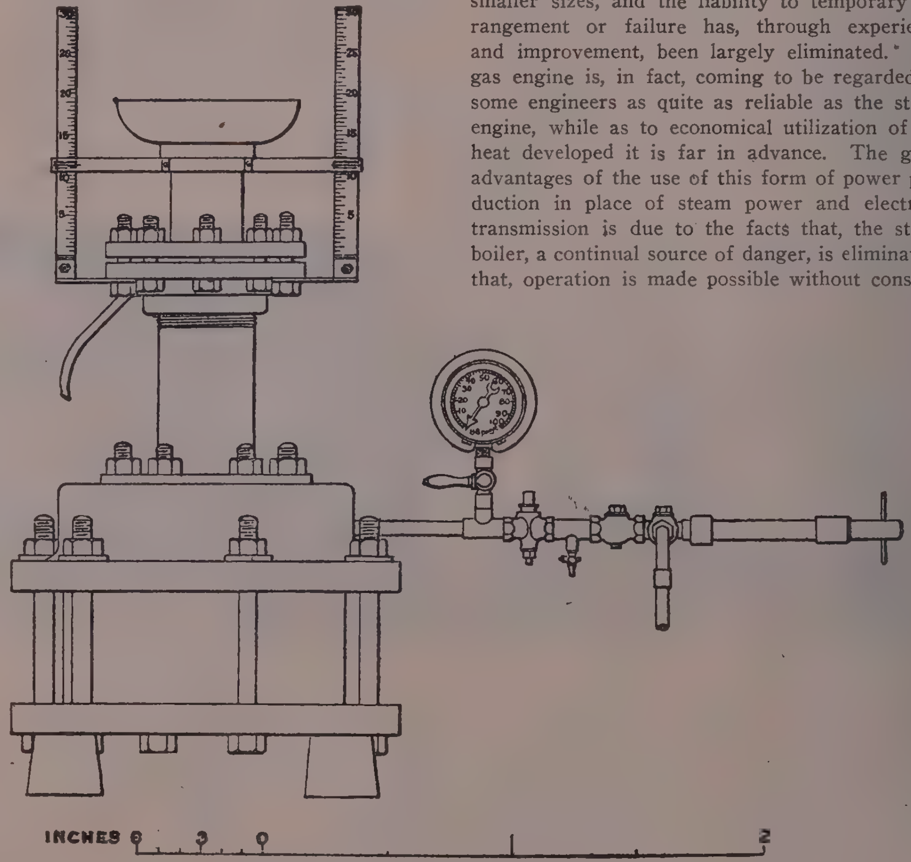
The cylinder of concrete on which the experiments were conducted was 13 in. in diameter and 6 in. in thickness, and was made in a heavy wooden mold, on March 11, of Portland cement and crushed gravel ballast, gauged in the ratio of 1 to 4. The cement used was a normal chamber-kiln product giving the following tests: Residue on a 50x50-mesh sieve, 1.0 per cent.; residue on a 76x76-mesh sieve, 4.0 per cent.; setting-time, 1 hour; tensile strength of briquettes made with 22 per cent. of water and shaken into a mold without pressure, 450 lb. per square inch after 7 days, and 600 lb. per square inch after 28 days; specific gravity, 3.16. The ballast was obtained from the bed of the River Test; the stones were passed through a 1-in. ring, and both stones and gravel were of varying fineness, thus considerably reducing the percentage of voids.

Table I.

Table Showing the Variation in the Amount of Water Percolating through 6 Inches Thickness of Portland Cement Concrete.

Days from commencement.	Duration of the experiment.			Pressure in lbs. per sq. in.	Flow in Imp. gallons per hour per sq. foot per lb. of pressure.	Per cent. of flow in the first experiment.
	h.	m.	s.			
1	2	2	35	36	0.000398	100.0
1	0	7	25	36	0.000387	97.3
1	0	5	0	60	0.000345	86.7
1	0	9	0	60	0.000191	48.0
2	1	16	40	24	0.000197	49.5
2	9	35	55	24	0.000172	43.2
3	13	0	0	34	0.000125	31.4
4	25	9	0	34	0.000109	27.4
4	9	21	0	34	0.000087	21.0
5	18	40	0	40	0.000090	22.6
6	24	10	0	40	0.000075	18.8
7	16	56	0	40	0.000066	16.6
7	2	53	0	40	0.000060	15.1
22	26	39	0	32	0.000014	3.5
23	10	19	0	32	0.000012	3.0
31	26	46	0	31	0.0000083	2.1
32	28	59	0	31	[illegible]	[illegible]
36	71	18	0	31	0.0000070	1.8
38	48	7	0	31	[illegible]	1.5
42	69	50	0	30	0.0000049	1.2
44	24	25	0	48	0.0000088	2.2
45	21	35	0	48	0.0000066	1.7
46	25	15	0	48	0.0000028	0.7

The concrete had a specific gravity of 2.23, corresponding to a weight of 140 lb. per cubic foot. It was removed from the wooden mold on May 18, and was fitted into a water-tight steel ring 13½ in. in internal diameter and 6 in. in height, forming part of the apparatus used in the experiments. In order to prevent side-flow best Swedish pitch was poured in small quantities, in a boiling condition, into the space between the concrete and the metal; and to obviate any tendency to leakage through the pitch, rings of best rubber, 3/16 in. in thickness, 12 in. in internal diameter and 14 in. in external diameter, were placed above and below the test-block and between the block and the planed faces of the apparatus. The test-piece was fitted on a heavy cast-iron ring of exactly 12 in. in internal diameter, connected by bolts and nuts to a heavy cast-iron shallow reservoir above, accurately turned to an internal diameter of 12 in. and provided with a heavy flange, in which, as in the lower plate, was recessed a lip, slightly beveled inwards, thus ensuring, with the rubber rings, already referred to, an absolutely water-tight joint. The reservoir was surmounted by a heavy cast-iron stem, recessed into it and making with it an absolutely water-tight joint; at the upper end of this stem was an easily-adjustable gland, consisting of a coil of rubber packing between two metal disks beveled internally, any water accumulating in the gland during an experiment being drawn off by a drip-tube. An accurately-turned steel plunger working through this gland, carried at its upper end a metal cap on which the loads were piled; and a pointer, rigidly attached just beneath the shoulder of the cap, slid over two fixed scales graduated in millimetre-divisions. The pressure on the upper face of the test-piece was recorded on a very sensitive Bourdon gauge fixed on a ½-in. pipe, the center of which was 1 in. above the upper face of the test-piece. The flow was measured by the descent of the piston, a drop of 1 millimetre being equivalent to a discharge of 6.205 cu. cm. of water, and the areas were so adjusted that a discharge of 1 cu. cm. per second was equivalent to a discharge of 1 imp. gal. per hour per square foot of test-piece exposed. The water used, which was taken from the corporation mains, was pumped from chalk, and was reduced from 18° to 6° of hardness, which was the hardness maintained throughout the experiments. The temperature of the air throughout the experiments varied between 12° and 15° Cent., but Mr. Baldwin-Wiseman did not introduce any correction for variation of the viscosity of the water with temperature, or for variation of the volume with pressure and temperature, as these did not appreciably affact the results.



Apparatus Used in the Experiments.

It will be observed, on examination of the results of the experiments shown in Table I, that, for a constant pressure of moderate magnitude, the flow at any time varies inversely as the time from the commencement of the experiments, but with a sudden change of pressure there is a corresponding variation in the flow. Towards the conclusion of the experiments, small stalactitic growths appeared on the underside of the test-piece, and the percolation was so slow that no water fell into the drip-tin, the water being evaporated from the underside of the test-piece as quickly as it percolated through. The results of the experiments appear to show that, at or near the surface of the concrete, and under a moderately high pressure, the water dissolves out some of the material of the concrete, but under the reduced pressure which prevails in the small pores at some distance from the surface the dissolved material is precipitated on the sides of the pores, reducing the flow and eventually checking it.

Internal Combustion Engines for Shop Power.

The use of internal combustion engines for shop power is meeting with increased favor and will probably become as extensive as the electrical transmission method now employed. The internal combustion engine has been brought to a high degree of development; particularly in the smaller sizes, and the liability to temporary derangement or failure has, through experience and improvement, been largely eliminated. The gas engine is, in fact, coming to be regarded by some engineers as quite as reliable as the steam engine, while as to economical utilization of the heat developed it is far in advance. The great advantages of the use of this form of power production in place of steam power and electrical transmission is due to the facts that, the steam boiler, a continual source of danger, is eliminated; that, operation is made possible without constant skilled attention, and that, power is produced more economically. These considerations have recently influenced a number of large industrial concerns toward the use of the internal combustion engine for shop power production, among which is the Baldwin Locomotive Works at Philadelphia, Pa. With this company, the handling of coal under city conditions had an important bearing on its decision. The Hornsby-Akroyd type of engine was selected for the service required. In recent tests made at the Baldwin works it has been found that the total cost of power production with this type of engine was 0.436 cents per brake horse-power hour, while the lowest cost for the steam engine observed was 0.786 cents per brake horse-power hour. The steam power equipment used in these tests involved a compound condensing steam engine and a boiler evaporating 8.05 lb. of water per pound of coal. The oil consumption of the internal combustion engine was 11.3 gal. per 100 brake horse-power hour, the specific gravity of the oil used was 0.908, so that a gallon weighs 7.57 lb.

A Single-Track Four-Span Reinforced-Concrete Interurban Railway Bridge.

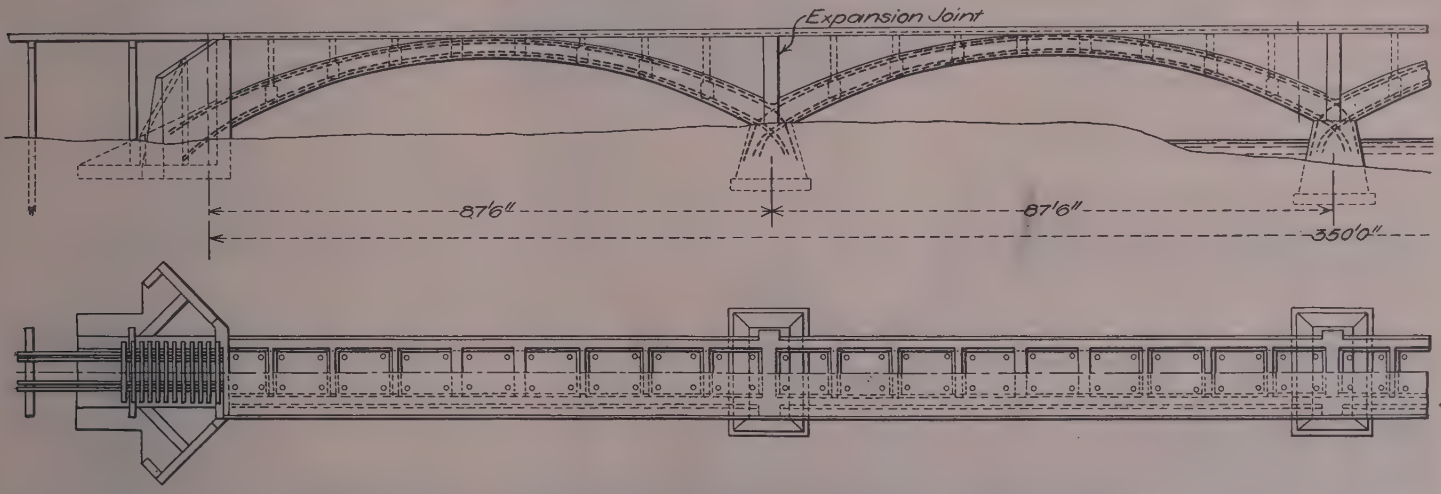
A single-track, reinforced-concrete bridge, 350 ft. long, comprising four arches of 81 ft. clear span each, has recently been completed on the line of the Elgin-Belvidere Electric R. R. over the Kishwaukee River about 2½ miles east of Belvidere, Ill. Each arch of the bridge has a circular intrados with a radius of 83.36 ft. and a rise of 10.5 ft., the piers being spaced 87.5 ft. apart on centers. The arches each have two longitudinal arch ribs, 8 ft. 10 in. apart on centers. These ribs are 2.5 ft. wide in their entire length, but vary in depth from 3 ft. at the crown to 4 ft. 6½ in. at the haunches of the arch. Each rib carries a 12-in. spandrel wall centrally disposed above the rib and built up to the horizontal plane through the extrados of the arch at the crown. For lateral stiffness, the two ribs of each arch are connected by eight transverse beams in each span, which beams are spaced at equal distances longitudinally along the arch. On these beams are 12-in. cross walls built flush at the top with the spandrel walls, and on the latter and the cross walls the floor slab of the bridge is carried. This slab, which is 6 in. thick and 14 ft. wide, has along each side of it a curb, 8 in. high, built to confine the ballast of the roadbed. Drain holes are provided in the floor slab at intervals. At each of the piers a pilaster is brought up on each side of the bridge to within 3 in. of the ledge of the floor slab to support poles carrying the trolley wire of the electric line. Seats are provided at the abutments for the connection of trestle approaches to the bridge.

The water level in the stream across which the bridge has been built is about 3 ft. below the springing line of the arches at low stages, while in extreme floods it rises to within 5 ft. of the intrados of the arch at the crown. The bridge was so designed that in its erection all centering and falsework could be eliminated, thus avoiding many of the difficulties entailed by the use of timber falsework and centering. The ribs of the arch ring were built in trough-shaped reinforced-concrete forms. These trough-shaped forms were divided into 17 sections for each rib, making each section 2½ ft. wide over all and 4 ft. 11 11/16 in. long at the bottom. These sectional forms were 3 ft. deep at the crown and 4 ft. 6½ in. deep at the haunches of the arch, the intermediate sections being graduated in depth between these two dimensions. The sides of the sectional forms were 3 in., and the bottom 4 in. thick, the ends of the trough section in each case being omitted. In these hollow sectional forms was placed, after they had been erected, the core or arch rib proper, which was 2 ft. by 2 ft. 8 in. in cross section at the crown and 2 ft. by 4 ft. 2½ in. in cross section at the haunches of the arch. The corners between the sides and bottom of each form were filled out with concrete, as shown in one of the accompanying illustrations. Each of these sectional forms was reinforced with ⅜-in. plain round steel rods placed 8 in. apart on centers longitudinally and 12 in. apart on centers transversely. The weight of sections varied from 1,500 to 2,200 lb.

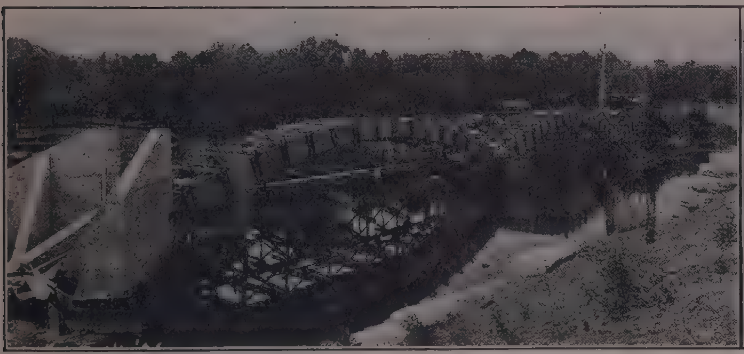
The sectional forms of the arch ribs and those for the transverse beams connecting the pairs of the latter in each arch were cast in a series of steel moulds. These moulds were built of No. 16 sheet steel stiffened by 3-in. steel channels and held in alignment by angle irons so arranged and connected that each mould could be knocked down and reassembled readily. In all, 136 sectional rib forms and 32 transverse beam forms had to be cast. Since each rib was symmetrical about the transverse center line of its arch, only nine different kinds of sectional forms were required for the ribs. One steel mould was sufficient for the transverse beams. By inserting loose wooden strips in the proper position between the inner and outer side pieces of the mould the variation in the depth of the rib forms was readily effected. The sectional rib forms were cast with the moulds standing vertically so that one voussoir face was against the working platform on which the mould stood while the other was flush with the top of the mould. The concrete in these rib forms was made very wet in the proportion of one of cement and three of sand, and filled the steel moulds thoroughly.

A hole was cast in the side of each sectional form at about the center of gravity of the latter and through these holes attachments were made for handling the forms after they had set. Those pairs of sectional forms in each of the arch ribs which are connected by the transverse beams had two rectangular openings cast in one side to facilitate this connection, the openings being made by fastening wooden blocks to the mould before the concrete was placed in the latter. The sectional forms hardened rapidly enough so that it was possible to remove the steel moulds 24 hr. after the concrete had been placed in them. When the sectional rib forms had all been completed they were numbered and stored at the site until required in the structure.

In the meantime two longitudinal rows of piles were driven on both sides of the bridge. These rows of piles were capped with 2x10-in. transverse plank braced in both directions by 1-in. boards. On top of the caps 6x10-in. stringers were laid, and on these stringers were placed two rails for a light tramway. A traveler spanning the site of the arches was mounted on these two parallel tramways. This traveler was fitted with two triplex blocks, each carrying a balanced beam made up of two 8-in. channels. At the proper time the sectional forms for the arch ribs were lifted to a platform on the temporary bridge by a guyed derrick. The corresponding sectional forms for the two ribs of each arch were then assembled in pairs and spaced the same distance apart transversely they were placed in the



Half Elevation and Half Plan of Four-Span Reinforced-Concrete Bridge.



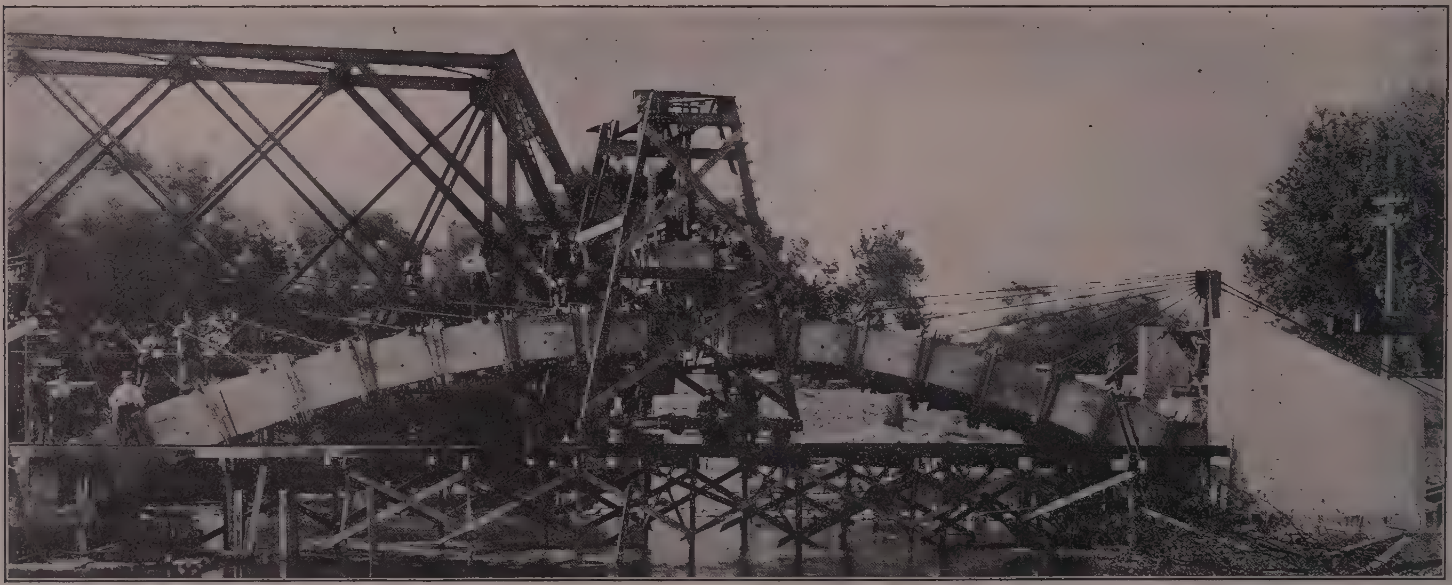
Sectional Forms for Four Arches in Position.

The Completed Structure with Forms Removed.

bridge. Two 4x6-in. timbers were placed transversely under each pair of sectional forms and bolted to two other 4x6-in. timbers placed transversely over the top of the pair of forms by bolts extending up through holes in the bottom of the forms. The pair of forms was thus firmly held in a lifting frame to which were attached hooks on the balanced beams carried by the triplex blocks on the traveler. The pair of forms would then be lifted into place in the two arch ribs and suspended there, as will be explained later. In this way the two ribs of each arch were erected simultaneously, and where a pair of rib forms were connected by the forms for a transverse beam the three were placed as a unit.

Derrick Lifting a Sectional Form.

The erection of the arch ribs was started simultaneously from each haunch, the first pair of sectional forms being supported at one end on the skewback and at the other end attached to a set of steel rods suspended from an A-frame at the end of the arch. One of these A-frames was set at the abutment and anchored back to the piers of the trestle approach to the bridge; the other A-frame for the first arch was set on the pier at the other end of that arch and anchored back to the second pier. The anchors for each A-frame consisted of 1⅜-in. round steel rods in 16-ft. lengths, connected at the joints by sleeve nuts. Turnbuckles were also provided on these rods so that the length of the anchors could be adjusted. A series of eight 1⅛-in. round steel rods, each provided with a turnbuckle, radiated from the top of each A-frame. These rods each supported two shorter 1-in. rods diverging to the sectional rib forms. The shorter rods were connected to U-bolts on the lifting frame carrying the assembled pairs of sectional rib forms, and supported the latter until the rib forms for one arch had been placed. In addition to the 4x6-in. timbers of the carrying frame in which the pairs of rib forms were assembled, two light wooden frames on each pair of forms were used to brace the latter. The second sectional pairs of forms were brought into position after the first had been placed, and the two sections of that pair supported at one end on the two sections of the first pair and at the outer end by the second set of rods attached to the A-frame. The connection between the sectional form was made by steel dowells placed in gas pipes built into the inside lower corners of the sections as the latter were cast, and in some cases by wooden blocks fitted to the inside of the section. In this manner the remainder of the sectional forms were placed in position and supported.


Placing the Keystones of the Two Sets of Sectional Forms of One Pair of Arch Ribs.

The keystone of each set of arch rib forms required no support other than that supplied by the two sectional forms adjacent to it. When the keystones had been placed, the rods attached to the A-frames supporting the sections of the forms were slackened and the two halves of each arch rib form allowed to close against the keystone of the latter. The entire set of sectional forms comprising those for each arch rib then became self-supporting and the A-frames with their anchors and rods were removed and used in a similar manner for the erection of the forms for the pair of ribs of the next arch. The lifting frames on each pair of sectional forms were removed as soon as the span was completed, but the bracing frames between the pairs of forms were left in position until the concrete had been placed in the sectional rib form.

In calculating the A-frame suspension rods the assumption was made that the sectional forms were supported at two points while being placed, which condition was actually established in the erection. The supension rods were also so adjusted that the joints between the various sectional forms were left open slightly at the top.

To insure the proper fitting of the keystone, the faces of the skew-backs on the piers were dressed before erection was started so as to make the angle between the face of the skew-back and the horizontal plane through the springing line slightly less than that calculated. In addition, the lower edge of the first sectional form was rounded, in order to keep the pressure from being applied too close to the edge, and in the joint a strip of sheet lead, ⅛ in. thick, was placed. The turnbuckles on the anchor rods made it possible to raise or lower half of the arch rib forms as a unit, the rotation taking place about a horizontal axis near the lower edge of the first sectional form, with an effect practically the same as if a hinge had been inserted in the joint at the skew-back. In this way the opening in the center for the keystone was adjustable, so no trouble whatever was experienced in fitting the keystone in place. After the removal of the A-frames and supporting rods, all the joints were found tightly closed and the alignment was perfect.

Table I.

Elevations and deflections observed at intrados of arch ribs at crown.

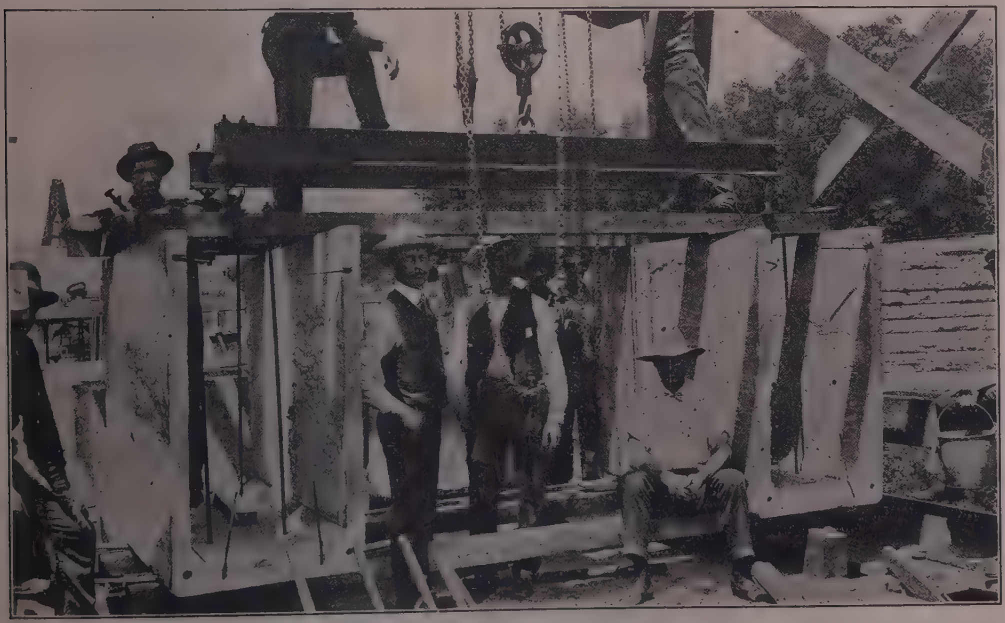
Span Number.		Elevations in ft. Before any concrete was placed.	Elevations in ft. After concrete was placed in ribs & beams	Elevations in ft. After concrete was placed in walls & floor	Deflections in ft. Caused by load of concrete in ribs & beams	Deflections in ft. Caused by load of concrete in walls & floor	Deflections in ft. Total deflection caused by dead load of structure.
One	North Rib	0.43	0.39	0.37	0.04	0.02	0.06
	South Rib	0.38	0.37	0.34	0.01	0.03	0.04
Two	North Rib	0.65	0.56	0.54	0.09	0.02	0.11
	South Rib	0.49	0.41	0.39	0.08	0.02	0.10
Three	North Rib	0.63	0.58	0.55	0.05	0.03	0.08
	South Rib	0.60	0.56	0.53	0.04	0.03	0.07
Four	North Rib	0.54	0.53	0.49	0.01	0.04	0.05
	South Rib	0.57	0.56	0.52	0.01	0.04	0.05
	Average	0.54	0.50	0.47	0.04	0.03	0.07

Elevations are 157 ft. +, but only decimals of feet are given.
Elevation at crown as per design, 157.46. Average actual elevation, 157.47.

The sectional forms for the ribs of all four arches were set in this manner before concrete was placed in any of them. This sequence of operations was necessitated by the fact that the piers would not have been stable under the unbalanced horizontal thrust caused by placing the concrete in the arch rib forms for one span before the skeleton arch of the adjoining span was erected. When the sectional forms for the ribs of the four arches were in place, the reinforcement was put in position in them. These ribs, together with the transverse beams were then filled by first putting in a 12-in. layer uniformly over them and then following up with the remainder of the concrete. The concrete in the arch ring and the transverse beams was made continuous by the openings through the inner walls of the sectional forms, and the entire construction thus rendered monolithic. After the arch ring had been completed in this manner, the spandrel walls and floor slab were built in ordinary wooden forms.

The erection of the arch rings was successfully completed in a comparatively short time, notwithstanding the fact that only unskilled labor was employed. An average of three days was required to erect the sectional forms for the arch ribs of one span, and to take down and re-set the A-frames with their anchorages. The sectional forms for the ribs of the last arch ring were erected in a day and a half. The largest number of sectional forms placed per day was 12 pairs. The entire super-structure of the bridge, including the arch rings, was completed in thirty days, which included three or four days' delay, caused by labor troubles.

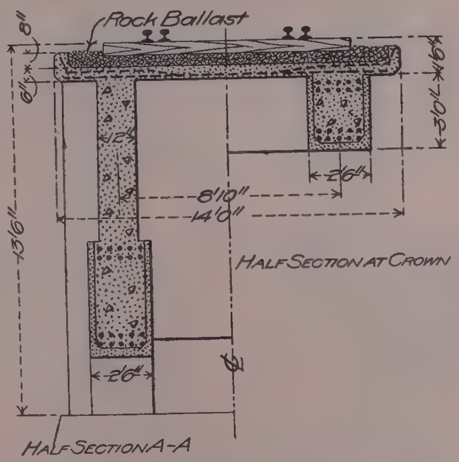
The sub-structure presents no features of special interest. The three intermediate piers were built of solid concrete, while the abutments were provided with buttressed wing walls, which, together with the walls and base slab, were all of reinforced concrete. With the exception of one pier and one abutment, the sub-structure was founded on gravel. At that pier quicksand was encountered, which caused a great deal of trouble and involved a delay of at least thirty days in the completion of the sub-structure. Some trouble was likewise encountered on this account at one abutment. A double-walled cofferdam was finally built around this pier and abutment and piling driven. The cofferdam was then sealed by depositing the concrete under water, after which the construction proceeded in the usual way.



A Pair of Sectional Rib Forms Assembled in a Lifting Frame.

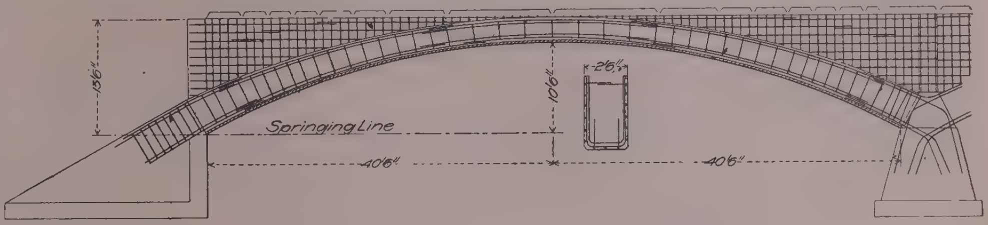
The mixture of concrete for the piers and abutments was 1:3:5 and for the super-structure 1:2:4. The bridge was built for a live load, consisting of a train of 40-ton interurban electric cars. Without considering temperature variations a unit compressive stress of 500 lb. per square inch and a unit tensile stress of 500 lb. per square inch is taken as the allowable stress in the concrete and 10,000 lb. per square inch in the steel. Considering temperature variations of 40° above and below 40° Fahr., the allowable unit compressive stress in the concrete is taken to be 650 lb. and the unit tensile stress 75 lb. per square inch, with 13,000 lb. per square inch as the allowable unit stress in the steel.

The stresses were figured for the arch fully loaded and half loaded and were combined with the temperature stresses to produce the maximum. Only the concrete and steel in the core of the arch ribs was considered effective, but since the sectional forms actually carry a considerable part of the dead load, the factor of safety of the bridge is higher than ordinarily obtained in reinforced-concrete construction.



Cross-Sections of an Arch.

The reinforcement in the arch rings consists of ten ⅞-in. plain round rods in both the extrados and intrados of each arch rib, with shear rods as shown in one of the accompanying illustrations. The ⅞-in. rods are about 30 ft. long and were spliced by overlapping 3 ft. These rods were also continued into the foundation. The reinforcement in the spandrel walls consists of ⅜-in. round rods spaced 12 in. centers, horizontally and vertically. The reinforcement in the floor consists of ½-in. rods, 5½ in. apart on centers, placed transversely near the bottom of the slab between the spandrel walls, every other rod being bent so as to be near the top in the outer part of the slab and to continue up into the curbs at the sides. There were also twelve ⅜-in. round rods spaced longitudinally throughout the floor and wired to the transverse rods.



Details of Reinforcement for One End Arch.

The deflection of the arch ribs was observed, first, immediately after the erection of the four arch rings and before any concrete had been placed in them; again, after the concrete had been filled in the sectional forms to form the solid ribs; and then, after all the concrete had been placed in the spandrel walls and floor of the super-structure. The results are indicated in Table I.

The deflection caused by the filling of the concrete in the ribs is somewhat variable, which is probably due to the fact that the joints between the sectional forms may not have all been closed up completely. After the ribs had been completed, the deflection caused by the additional weight of concrete in the spandrel walls and floor is very regular. The total deflection in no case exceeds the normal, and the average elevation of the eight ribs at the intrados of the crown, after the completion of the structure, deviated only 1/100 of a foot from the calculated elevations.

The advantage of this type of construction is claimed to be the elimination of centering, which is always a large portion of the total cost of a concrete bridge, and in deep and swift rivers makes construction not only unduly expensive, but dangerous and hazardous. While for the sake of expediency in this bridge a traveler and a temporary track were used for erecting the reinforced-concrete forms, according to the designers, a cableway will be used in future work. With the latter arrangement it is believed by them, concrete bridges can be built across streams of practically any character.

The outfit required to construct a bridge of this type involves a larger first cost, but after this outfit has once been acquired, it is said to enable the construction and erection of the sectional forms at very little cost and at remarkable speed. The steel moulds used on this bridge could be used, it is understood, in constructing a series of arches ranging in length from 25 to 100 ft., and can be used repeatedly. Where a series of bridges is to be built at one time, this feature attains greater importance in reducing cost.

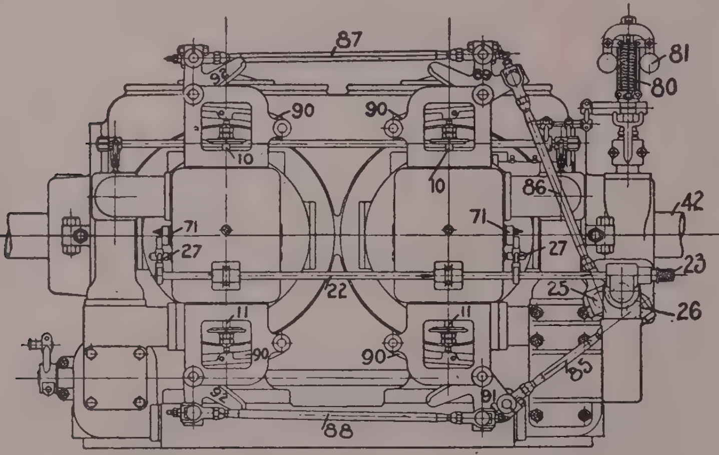
The Elgin-Belvidere R. R. is under construction by the Arnold Co., Mr. Bion J. Arnold, president, of Chicago. The bridge was designed and built by the Strauss Bascule & Concrete Bridge Co., of Chicago, Mr. J. B. Strauss, president. The work was executed under the immediate supervision of Mr. K. Hojgaard, engineer for that company, and Mr. Geo. Weston, engineer for the Arnold Co.

ALUMINUM-COATED IRON SHEETS are now manufactured in considerable quantity and will probably be used in place of galvanized iron under certain circumstances, as they have been found very durable on long exposure.

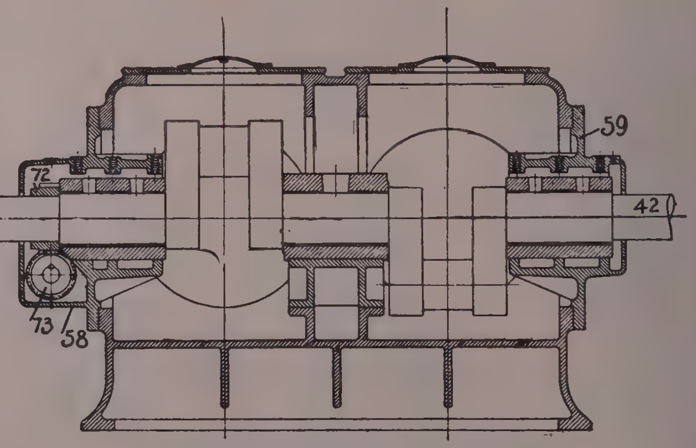
A New Design of Gas Engine.

A new design of internal combustion engine, which is intended to combine the most advanced practice in engines of that type with an extensive experience in the construction and operation of prime movers, has been perfected recently by The Buckeye Engine Co., of Salem, Ohio. The new engine is of the two-cycle, single-acting, scavenging type, and is adapted for operation with all classes of fuel both liquid and gases, although it has been designed with special reference to economical operation when using suction producer gas. The new engine will be built in the design illustrated below in sizes from 25 to 500 brake horsepower, while for powers above the latter capacity it is proposed to build engines of the double-acting type in either the two cycle, or the four-cycle construction.

The new two-cycle single-acting design illustrated herewith, involves an interesting duplex construction, with cranks 180 deg. apart and in addition to the two motor cylinders, an arrangement of cross-head cylinder on either side which serves both for fuel and air pumps for the scavenging process. The result of this arrangement is that with the two-cycle construction, two working impulses are given to the crank shaft per revolution. Another advantage is that, as the cranks are exactly opposed and connected to pistons of equal weight, the engine requires no counterweighting and turns at a nearly perfect running balance.



Rear Elevation Showing Valve Mechanism.



Cross-Section Through Crank Case.

The details of the new engine are illustrated in the accompanying sectional drawings, from which its operation may be readily explained. The most notable feature of the new construction is that instead of fitting the connecting rod to a cross-head bearing inside the trunk piston, the latter has a piston rod connecting through a stuffing box to a cross-head consisting of a piston working in an auxiliary cylinder next to the crank case; this arrangement results in a very rigid "straight-line" construction and also provides the fuel and air pumps with a minimum of complication. The main piston has a spherical head end for the reduction of cylinder clearance, while on the trunk, or inner side, it is made to serve as the fuel pump, by means of a water-cooled stuffing box which encloses the piston rod and projects into the interior of the trunk piston. Again, the space between this stuffing box and the cross-head piston is made to serve in a similar manner as an air pump. The connecting passages of both the air and fuel air pumps, as well as the arrangement of cylinder valves, are shown in the accompanying drawings.

When the engine is in operation, the piston in the crosshead barrel (46) furnishes air at a pressure of about 8 lb. per square inch for scavenging the combustion chamber (1) while the piston in the opposite crosshead barrel (45) compresses air at the same pressure for scavenging the combustion chamber in cylinder (43). Assuming the longitudinal sectional view to show a section through the right-hand engine, and the piston (2) just uncovering the ports (14), then compressed air furnished by the air pump in the barrel (46) enters the combustion chamber (1) through the valve (11) and the port (19) forcing out the spent gases. The combustion chamber at A-A is circular in section and conical in form, which assists in spreading the scavenging charge evenly behind the spent charge. Somewhat later, valve (10) is caused to open, admitting into the cylinder a charge of explosive mixture which has been compressed in the chamber (3) in front of the piston (2). It is thus seen that each motor piston compresses its own explosive mixture, but receives its compressed air for scavenging from the opposite engine. Since the valves (10) and (11) are independently driven the relative timing of the admission of the scavenging air and the mixture is under perfect control.

When the piston (2) has covered the ports (14) on the return stroke the compression begins in the motor cylinder, providing the valve (10) has closed. The compression being complete, the charge is fired in the usual way, firing occurring alternately in the two motor cylinders.

One of the detail drawings shows a side elevation of the right-hand engine with a section through the piston valve body (30) wherein are the two balanced piston valves (62) for admitting a mixture to the gas pump in the cylinder (44), and (63) for admitting a charge of air into the port (69) which is in connection with the air pump in the barrel (46). The chamber (68) communicates with the atmosphere through the sub-base of the engine. The two valves (62) and (63) serve only as admission valves to their respective pumps. The pumps discharge their contents through the poppet valves (10) and (11), as previously described.

A characteristic of this engine is that so much sought for by most gas engine designers, namely, constant compression and constant mixture at all loads combined with quick and certain response to sudden change of load. The quantity of air passing through the air pumps is constant at all loads, but the amount of air and gas handled by the fuel pumps varies according to the load carried. In some two-cycle engines a governor controlled throttling valve regulates the amount of fuel admitted into the fuel pump. For example, the governor might be arranged to throttle the supply to the chamber (67); in this case a sudden drop from full load to no load would find a full charge of fuel that would have to be consumed before responding to the change in load. In such an engine the work done in the fuel pumps would vary with change in load but the response to the change would be slow. Secondly, a throttling valve might be located at (83) to control the amount of mixture passing from the fuel pump to the combustion chamber. In this case a response to sudden change in load would be quick but the fuel pump would do equal work at all loads and thereby diminish the efficiency of the engine at low loads.

The system devised and used on this engine accurately combines the good qualities of the two preceding systems and eliminates their defects as follows: When the piston valve (62) uncovers the two ports (64), the fuel is sucked into the chamber (61) and from there through the balanced throttle valve (60) into upper chamber (61), which connects with the chamber (18), the latter being, as previously stated, part of the compression chamber of the fuel pump (3). The amount of fuel passing through the valve (60) depends on its position, this being directly controlled by the governor through the rod (83), the shaft (32), the two rods (33) and (34). When the compressed fuel flows from the pump cylinder (3) to (831) and the combustion chamber, it must return through the valve (60), or retrace through the same way it entered. Thus, clearly, the valve (60) controls the amount of fuel entering the gas pump as well as the amount flowing from the gas pump to the combustion chamber. This combination of the fuel piston valve (62) and the throttling valve (60) results in an economy of pump work and quickness of action which cannot be excelled by any other method whatever.

There are two sets of igniters in each combustion chamber, but only one is shown. The shaft (22) which operates the igniters is driven by the lay shaft (36) through a pair of spiral gears, one of which is keyed to the shaft (36), the other is splined to the shaft (32). An adjuster (23) is provided for shifting the splined gear on shaft (22), thus advancing or retarding time of ignition of all igniters. The engines are also provided with jump spark ignition apparatus, and are so fitted that the change from one system to the other can be made in a few minutes.

For starting the engine a balanced air valve (9) operated by hand for admitting compressed air is provided. This may also be operated automatically from the lay shaft when desired. The valves (10) and (11), together with the cages (12), are, as well as the corresponding set on the other cylinder, in exact duplicate and interchangeable. Only relatively cold gases pass through these valves so that they are assured of long life.

The temperature of the ingoing gases in all internal combustion motors should be low. Referring here to the fuel pump cylinder (3), it will be seen that the water in the jackets (16) and (41), which is kept as cool as possible, will tend to keep the temperature of compression low. The head (4) has on it a water-jacketed sleeve (41), which is babbitted and bored out, and on account of its considerable length, the speed of reciprocation and low pressures on either side, there is no leakage through the sleeve and it is subject to no wear whatever.

The connecting rod has, at its wrist end, a wedge adjustment, and at the crank end it is of the marine type. The main bearings contain babbitted shells of ample area and have means for alignment and taking up water. In this type of engine, owing to the action on each connecting rod of the combined air and fuel pumps and motor piston, it makes no material difference whether the wrist and crank pin bearings are carefully adjusted since the pressure is always in the same direction and therefore the engine will run quietly should even all the boxes be loose. The crosshead pistons and main bearings are oiled by splash lubrication, while the spiral gears driving the lay shaft are immersed in oil.

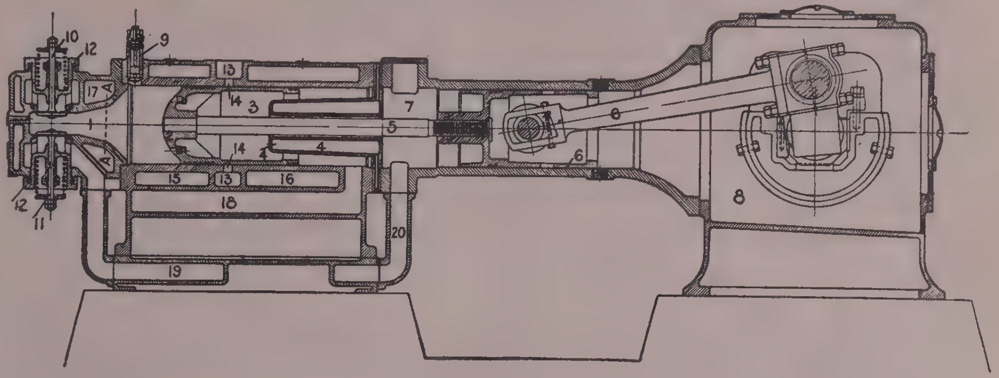
The accompanying engravings illustrate a 9x12-in. engine of this design which has been running over a year on natural gas and has shown some interesting characteristics. It has been found that in one end of the combustion chamber the mixture is unquestionably constant at all loads, and also the ignition is always certain from full load to no load. On account of the combined cooling action of the water jackets and the scavenging air, a higher degree of compression can be carried than in four-cycle engines, unless the latter are kept uneconomically cool.

Another important feature is that expansion can be carried to any point whatever, while with the ordinary four-cycle design the amount of expansion is proportional to the original volume admitted. In this engine the fuel valve can be so set that the compression may begin, say at half stroke, in which case the expansion would be equal to twice the volume admitted. In the smaller sizes this feature will not be made use of, but in the larger sizes an increased thermal economy is thus available.

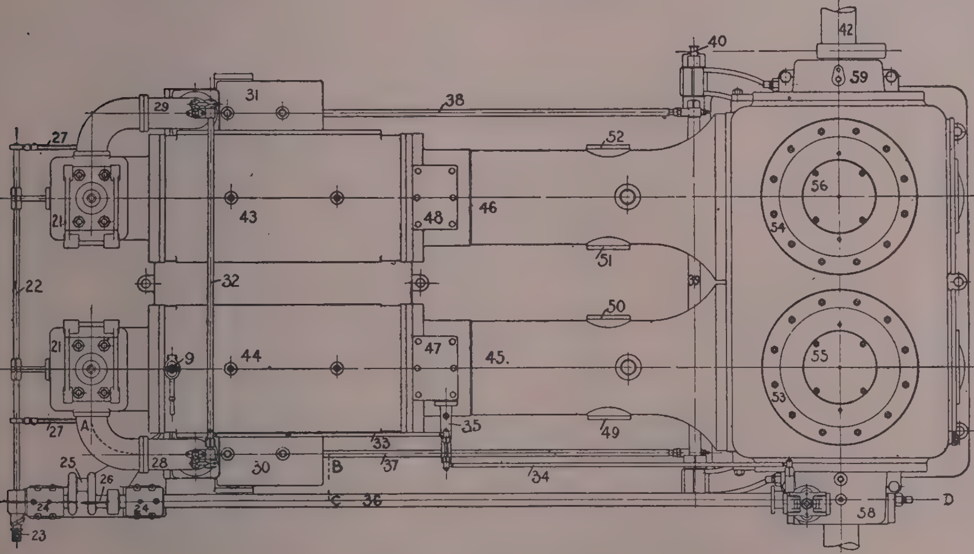
Suction producers require a variable amount of draft in their operation due to varying conditions of the fuel bed, for which purpose this engine offers great advantages, since the fuel pump has ample capacity to supply the engine with fuel no matter what the draft may be—within wide limits—and a producer can in emergencies be forced beyond its rated capacity without seriously diminishing the power of the engine.

This engine is also self-cleaning, thereby making it unnecessary to take off heads and valves for cleaning. All dirt is blown out and, in practice, it has been found that after many months of continuous service on suction producer gas, the combustion chamber was absolutely clean. This feature is valuable when using jump-spark ignition since there will be no trouble due to fouling of ignition plugs and the consequent destroyed insulation. The construction also permits, if so desired, a super-compression to be obtained whereby the main compression begins at from 3 to 5 lb. above atmospheric pressure; this increases the capacity of the engine with a possible slight decrease in thermal economy.

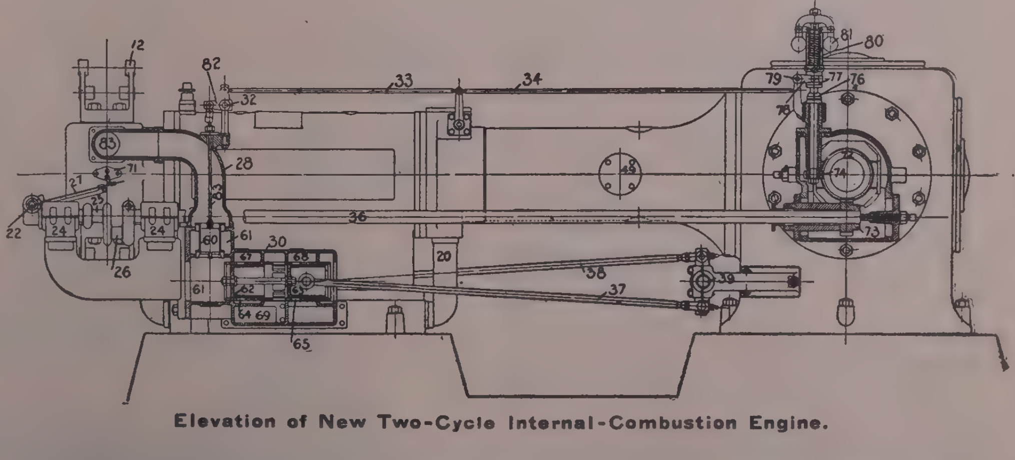
The experimental engine of about 75-h.-p. capacity, with cylinders 9 in. in diameter by 12 in. stroke, has been found by repeated tests to develop a mechanical efficiency over 85 per cent., the pump work being included as friction. The explanation for this high mechanical efficiency lies in the co-ordination of pump and motor cylinders, no loss of wire drawing through valves due to ample valve areas, the positive operation of all valves, the balancing of all valves and the absence of stuffing boxes.



Longitudinal Section of New Two-Cycle Engine, Showing Valve Construction.



Plan of Engine, Showing Arrangement of Valve Gear.



Elevation of New Two-Cycle Internal-Combustion Engine.

Heating and Ventilating Equipment of the Carnegie Branch Library No. 1, St. Louis, Mo.

The new Carnegie branch library No. 1, now approaching completion at St. Louis, Mo., possesses a complete mechanical plant, particularly in reference to its heating and ventilating equipment. This equipment embodies an important departure from general practice in that the air supplied is washed and thus all solid matter in suspension and noxious gases removed before being delivered through the duct system to the several rooms. After the air has been washed, the free moisture is taken out, leaving only such portion of it as is necessary to keep the humidity of the atmosphere at the proper point.

The new building is a single-story and basement structure, 58 by 105 ft. in plan, and is built of brick and stone. The main floor is covered by a skylight, but the window exposure is, in general, comparatively small. The system of heating depended on is, in general, the indirect, which is supplemented in the vestibule and exposed places only, by direct radiation. The heating for the indirect system is accomplished by means of live steam supplied at a pressure limited to 50 lb. per square inch for a maximum. The general arrangement of the indirect apparatus is shown in the accompanying basement plan. The air intake for the system is located at the sidewalk level in front of the building and has a sectional area of 24 sq. ft.

The blower used in connection with the system is a 71-in. steel plate fan which is direct driven by a 7½-h.-p. General Electric 500-volt, variable-speed motor. The blower wheel has a 35¾-in. face, and is built up on two spiders consisting of heavy steel tees, cast into heavy cast-iron hubs which are keyed to a shaft 2⅜ in. in diameter. The latter runs in heavy ring-oiling self-aligning bearings on either side of the fan. The housing of the blower is of No. 10 steel plate securely braced with angle and tee iron and provided with a horizontal outlet at the top. The fan wheel runs at a maximum speed of 250 r.p.m. without noise or vibration, at which speed it has a delivery capacity of 18,000 cu. ft. per minute.

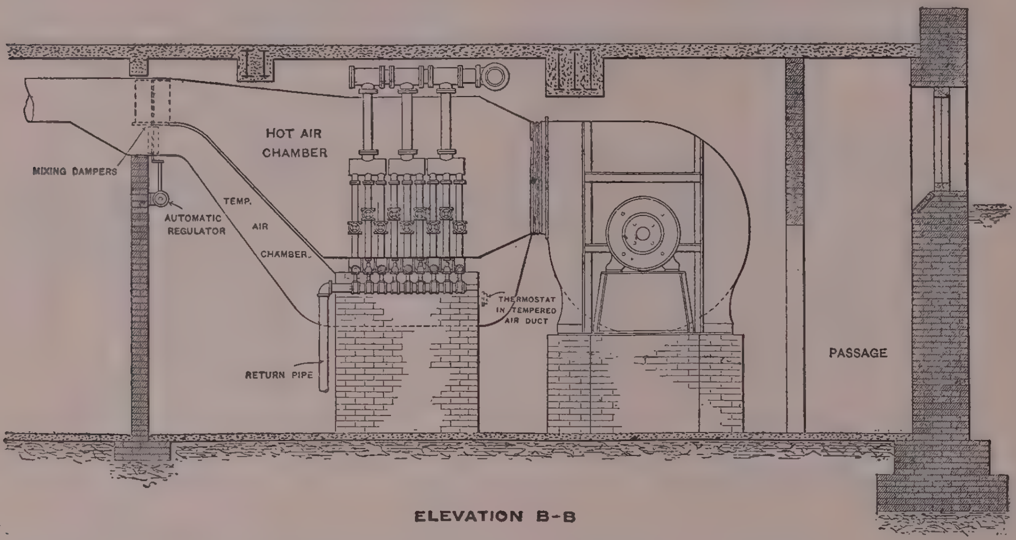
The tempering coils are five in number, each measuring 6 ft. 10 in. high by 7 ft. long. The sections of this heater contain 2,290 ft. of 1-in. pipe. The main heating coils are of a pattern similar to the tempering coils and are built on nine bases containing in all over 4,000 ft. of 1-in. pipe. The base connections for the heating and tempering coils are staggered on 2½-in. centers, and steam is supplied to the base of the coils through a 5-in. pipe from a boiler in the adjoining room. In addition to the factory test, each section was tested hydraulically after shipment to the library building to find any leaks that may have developed due to handling and also after the coils were in place and connected, they were subjected to a service test at 50 lb. maximum steam pressure. The tempering and heating coils are housed in casings of No. 12 steel plate, securely stayed and stiffened with angle and tee iron.

The control of temperature of the air delivered to the rooms is secured by two vertical leaf dampers at the entrance to each duct, which dampers are automatically controlled by thermostats placed in each room of the building. Both dampers are placed on the same vertical shaft with their planes at right angles, so as to act alternately; hence if a room is too warm, a single movement of the thermostatic control motor serves to cut off part of the hot air supply and allow more of the tempered air to pass. In this way the amount of air being delivered to the rooms is constant, and hence uniform ventilation is assured at all times. The ducts leading from the mixing dampers to the registers or diffusers and risers are of galvanized iron of No. 26 gauge. The ventilation or removal of the foul air from the building depends on natural draft, aided somewhat by the pressure caused by the blower constantly supplying fresh air.

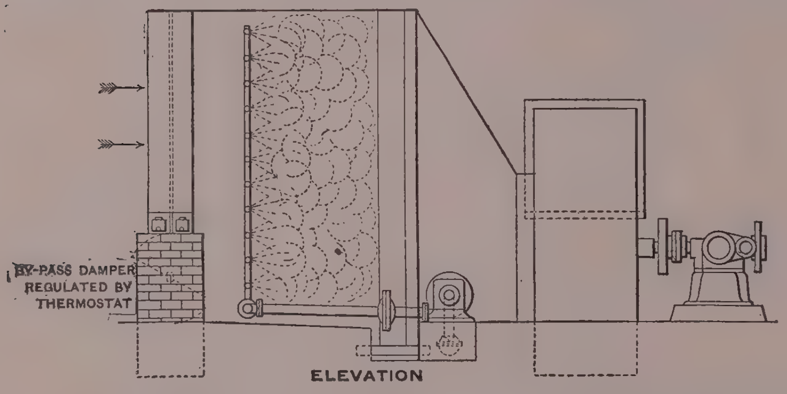
Owing to the large amount of soot and dust in the atmosphere of St. Louis, and to the fact that the air inlet for the ventilation system was necessarily at the sidewalk level, where street filth and disease germs are liable to find their way through the grating, it was found desirable to thoroughly cleanse the air before allowing it to pass into the library rooms, where otherwise much damage would be done. In the past the only undesirable feature of fan ventilation for libraries has been the tendency to increase rather than decrease the amount of dust and disease germs entering the building; but this objection is overcome by the introduction of the air-washing equipment. Instead of the usual type of coke and cloth screen cleaners, the air washer was found much more economical of space and operation cost. It also included the solution of the next problem, namely, that of allowing the air to pass with a proper and constant degree of humidity.

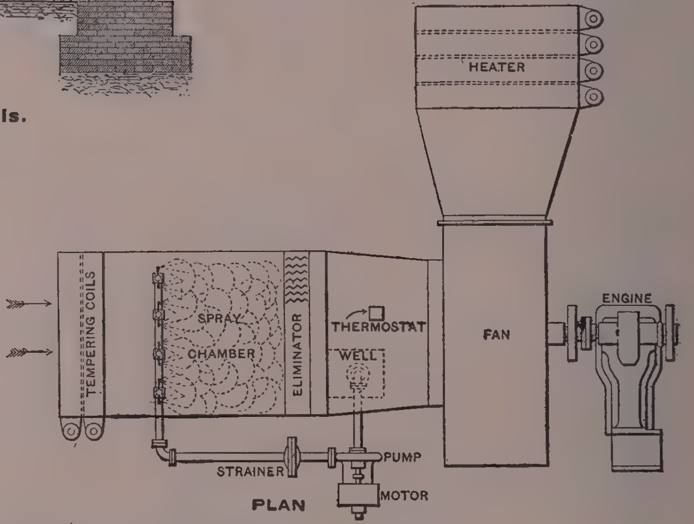
The air is delivered to the washer after first passing through the tempering coils, which serve during cold weather to raise the temperature of the incoming air to 70° or 80° Fahr. The washing is then accomplished by water projected in the direction of the air travel, from nozzles spaced closely enough to fill the entire chamber with the water spray. The type of nozzle is special, in that the water is atomized by centrifugal action due to its own path in passing from the nozzle, which delivers 1½ lb. of water per minute. So effective is this atomizing action that the spray is practically invisible. The spray water is recirculated, being supplied by a small brass centrifugal pump operated by a 1-h.-p. motor. The nozzles are easily taken apart by hand to remove any scum or deposit that may collect.

The air, after passing through the spray, encounters the eliminator, a system of vertical baffle plates so designed as to completely remove the free moisture from the air and allow the air to retain only the approved amount of humidity. The first portion of the baffle plates, present to the air a water-covered surface, contact with which completely removes every remaining trace of solid matter which may have been in the air. The second portion of the plates is supplied with vertical gutters to prevent the water from being blown across their surface. Contact of the air with these gutters serves to remove the free

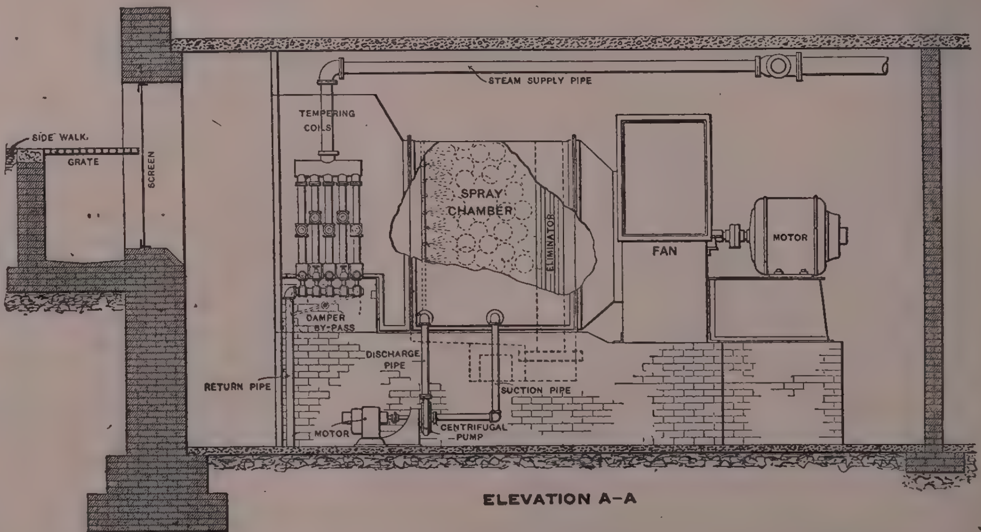


Heating Equipment of the Carnegie Branch Library, St. Louis.





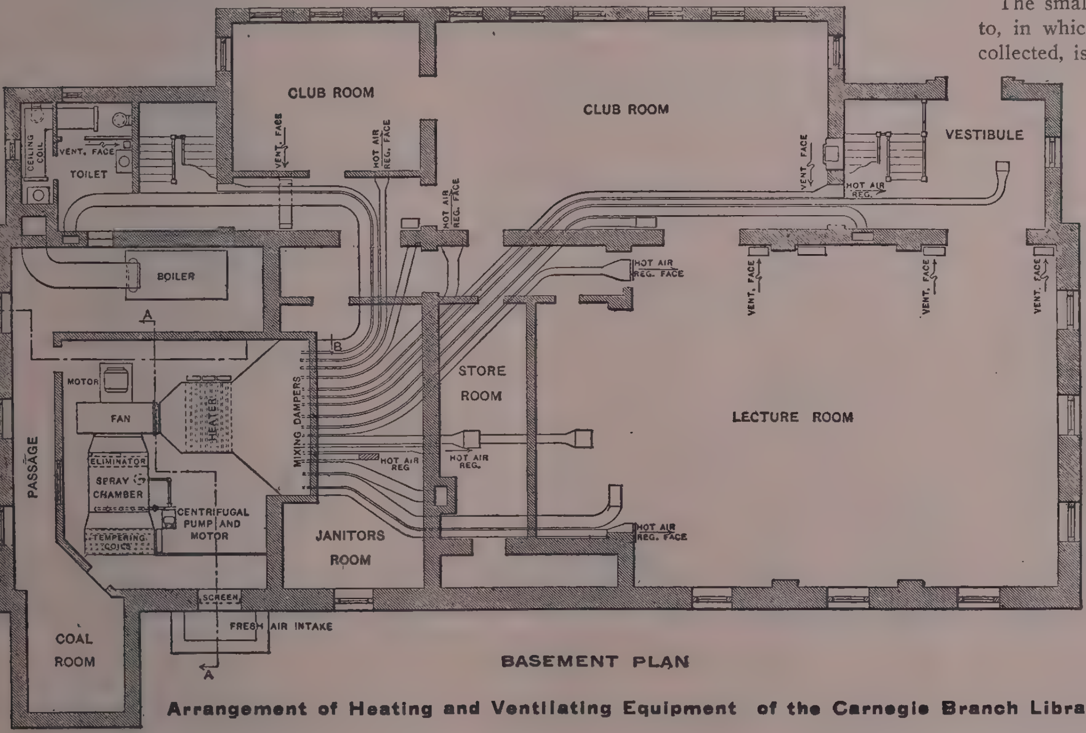
Details of the Air-Washing Apparatus of the Ventilating Equipment.



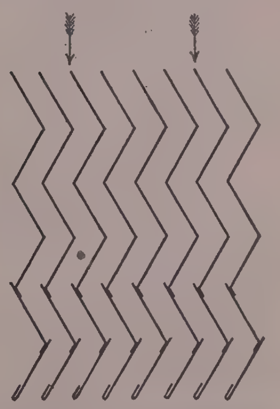
The Intake, Tempering Coils and Air Wash.

moisture, yet the resistance of the baffle plates to the passage of the air is only one-tenth of 1-in. water pressure. The plates are galvanized and riveted and are hence non-corroding, and owing to the method of forming they are extremely rigid without being of excessive weight. The water runs from the baffle plates to a settling tank below, where there is provision for removing the collected dirt.

Between the eliminator and the blower is located a thermostat which is adjusted, till, by trial, it results in securing the proper amount of humidity in the air after passing the heating coils and the mixing dampers. The action of this thermostat is to regulate the temperature of the spray water, when the temperature of the air between the eliminator and the fan changes. It has been found after extensive experiment that the humidity or amount of moisture that a body of air will absorb is dependent, not so much on the temperature of the air as on the temperature of the water in contact with it, and hence this scheme of regulating the humidity. It may be added that the temperature of the water is raised or lowered by the thermostatic control acting on a steam jet playing into the water supplied to the nozzles.



Arrangement of Heating and Ventilating Equipment of the Carnegie Branch Library.



Details of Eliminator Baffles.

A casing of galvanized iron stiffly braced and closely riveted and soldered, so as to be water tight, is provided for the washer and eliminator, which are capable of handling 18,000 cu. ft. of air per minute. In the casing is located a watertight door and a panel fitted with a heavy plate-glass window for inspection of the air washer while in operation. The rapid evaporation requires a supply of fresh water, which is provided by a float valve in the settling tank that keeps the water always at the same level.

The washer also serves an important purpose in cooling the air during the summer, for which purpose the contract calls for the guarantee that the final difference in the temperature of the water sprayed and the air, shall not exceed 20 per cent. of the initial difference in their temperature.

The engineers and power plant contractors for this installation were the Elliott and Barry Engineering Co., St. Louis, Mo.; the blowers, heating coils and air washing and humidifying equipment were built by the Buffalo Forge Co., Buffalo, N. Y. The architect of the building was Mr. Theodore C. Link, St. Louis, Mo.

Raising of Water by Compressed Air at Preesall, Lancashire.

Some experiments with the air lift, carried out at the works of the United Alkali Co., at Preesall, where large quantities of water are raised from wells by means of compressed air, are described in a paper by Mr. James Kelly, recently published by the Institution of Civil Engineers.

The water-bearing strata are found at a depth of 80 to 100 ft. from the surface. In the early days of the company's operations sufficient water to meet its requirements was obtained by sinking wells into the gravel, but as the business developed two wells were sunk into the red sandstone, and were fitted with lifting-buckets and the necessary engines. The cost of maintenance of this plant was very high, due, doubtless, to some extent, to the pumps being forced in order to cope with the increasing demand for water, but chiefly to sand in the water cutting up the buckets, valves and working-barrels. Having regard to this, and other peculiar circumstances of the case, namely, the necessity of raising water over a considerable area, owing to the low yield from any one well, and the desirability of having some system free from any great liability to stoppage or breakdown, it was decided to adopt the compressed-air method for the next well sunk. This well is 450 ft. in depth and 12 in. in diameter at the bottom, and is fitted with a lining-tube going down to the rock-head, and an air-tube 6 in. in diameter, with a delivery-tube 4 in. in diameter inside it, the effective cross-sectional area of the air-tube being thus the area of the annulus formed between it and the delivery-tube. For working this well a small Ingersoll-Sergeant air-compressor, already owned by the company, was modified to suit the conditions of work at Preesall.

The results of some preliminary working of the system were considered so promising that the installation of a larger plant capable of fulfilling all the requirements of the works was decided upon. There are now four wells equipped for raising water by compressed air. The air-pipes have been designed to reduce loss by leakage or friction as much as possible, and the water flowing from the wells descends by gravity to a small brick reservoir. In one well the air-pipe passes down the well outside the delivery-pipe and is connected to it at the bottom. A new air-compressing plant was installed, consisting of an Ingersoll-Sergeant cross-compound air-compressor, having both the air- and the steam-cylinders compounded. The speed is regulated by a governor in the usual manner, and also by the pressure of the air as it leaves the high-pressure air-cylinder. The air-cylinders are water-jacketed, and the air from the low-pressure air-cylinder is delivered to an inter-cooler, from which it passes to the high-pressure air-cylinder. The steam-cylinders are 20 and 32 in. in diameter, and the air-cylinders 28.25 and 16.75 in. in diameter, the stroke being 24 in. The percentage of clearance in the air-cylinders is very small. The air is compressed to 35 lb. per square inch in the low-pressure cylinder and in the high-pressure cylinder to a pressure ranging between 80 lb. and 120 lb. per square inch, according to the depth of submersion of the delivery-pipes in the well or wells under operation. The machine is rated very much higher than the present requirements, so that, should a greater quantity of water be required at some future time, more wells can be sunk and operated from the same station. The small compressor first installed is kept as a spare machine.

The small brickwork reservoir already referred to, in which the water raised from the wells is collected, is divided into two equal compartments by a wall which is lower in the middle than at the sides. The water flows into one compartment of the reservoir, where the sand in it is allowed to settle; it then flows over the weir formed in the centre of the mid-wall, into the other compartment, which contains the suction-pipes of the force-pumps employed for raising the water to the works-reservoir.

In making tests of the equipment the water raised was measured in the reservoir. The compartment containing the suction-pipes of the force-pumps having been pumped nearly empty, the force-pumps were stopped. The reservoir was then allowed to fill up to a given level with water from the air-lift, the time occupied in the operation being noted. Meanwhile the level of the water in the well had been determined, in order to ascertain the "lift," or height of delivery. In the engine-room, indicator-diagrams were taken from the steam- and air-cylinders, the speed of the compressor was registered by means of a stroke-counter, and the air-pressure was read off from the gauge. Thus all the data necessary for calculating the results were obtained.

The first series of experiments was made in January, 1903. At that time the delivery-pipe of this well was carried 40 ft. above the surface of the ground, the original intention having been to run the water by gravitation to the works-reservoir directly. The total length of the delivery-pipe was 433 ft., and its area was 12.56 sq. in.; the area of the air pipe was 16.2 sq. in. The results, which are recorded fully in the paper, show that with the compressor working between 17 and 29.6 r.p.m., the number of gallons of water raised per minute was practically the same at all rates of working; that, as the rate of working was increased, the volume of air used per cubic foot of water raised increased very rapidly, and the efficiency fell off at a proportionately rapid rate. This set of experiments indicated the futility of forcing the output of a well, and also that the series had been started at too high a rate of working, and that increased efficiency might be obtained at lower rates of working. In order to obtain, if possible, a higher efficiency, the 40-ft. length of delivery-pipe projecting above the surface of the ground was removed and a 60-ft. length was taken out of the well, reducing the length of the delivery-tube to 323.5 ft. This made the ratio of height of delivery to "dip" (depth of submersion) approximately 1:1.5. The highest efficiency obtained in this series was 24 per cent., which, although still low, was a decided improvement on the results obtained in the first series, which were from about 10 to 20 per cent. The efficiency is the ratio of the work done in raising water to the work indicated in the air cylinders.

The highest efficiency was obtained at the lowest rate of working, the volume of free air used per cubic foot of water raised being 8 to 13 cu. ft.

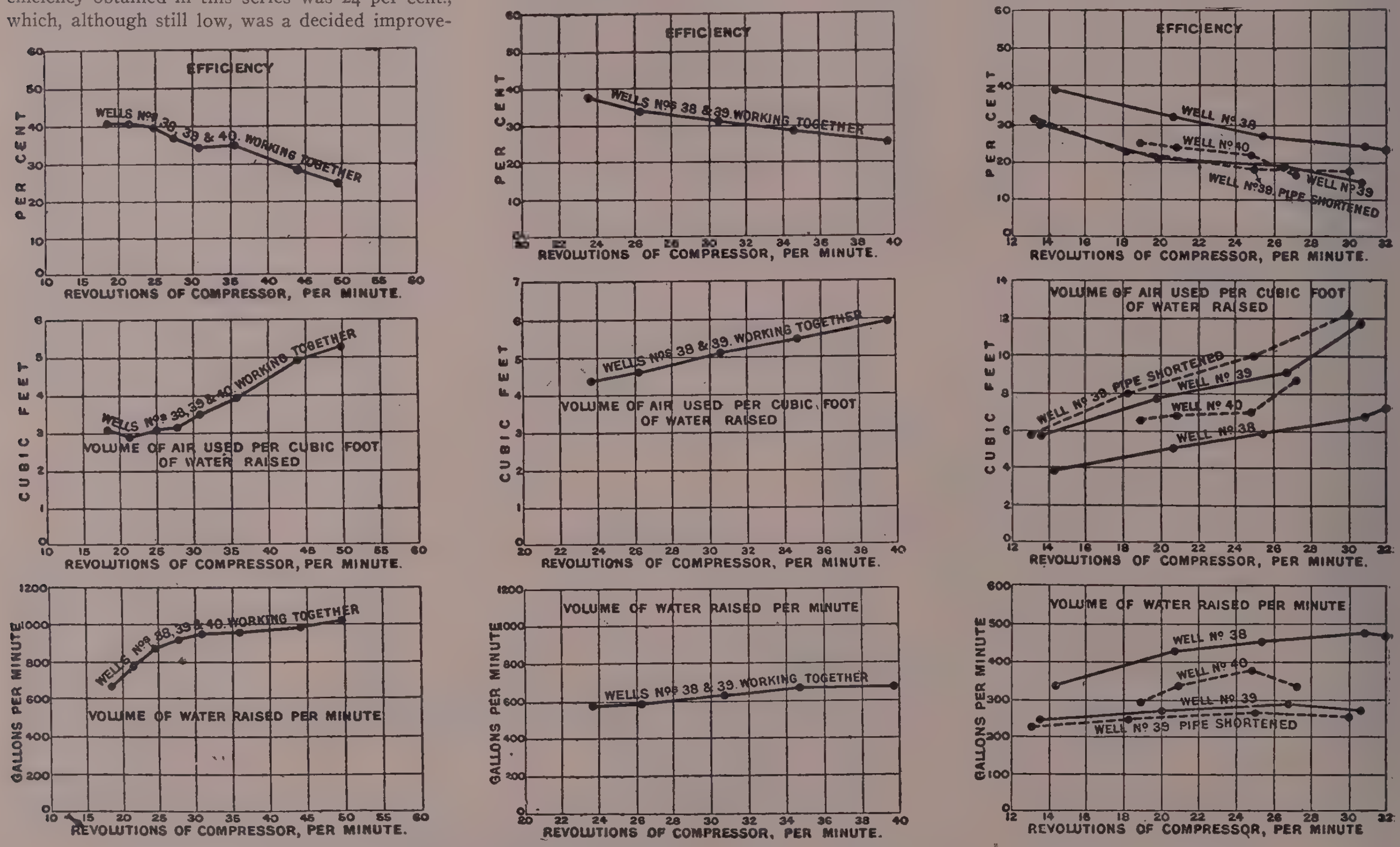
The next series of experiments was made in April, 1903, on a well 350 ft. in depth and of the same diameter as the first, 12 in. It has, however, larger pipes fitted into it, the delivery-pipe being 5 in. and the air-pipe 7 in. in diameter. The air-pipe has thus an effective area of 18.75 sq. in. In the first four experiments the delivery-pipe was 309.5 ft. in length, and on working out the results it was found that the ratio of dip to height of delivery was approximately 1:1. This was altered by adding a 30-ft. length to the delivery-pipe, after which two more experiments were made. The lengthening of the delivery-pipe had a marked effect on the efficiency, the efficiency in experiment No. 1 being 29.3 per cent. and in experiment No. 5, 34.3 per cent., the same proportionate difference being observed between experiments Nos. 2 and 6, indicating the importance of the ratio of dip to height of delivery being kept nearly equal to 1.5:1. The best feature of this series of experiments was the increase in the efficiency. The difference in area of the delivery-pipes in these two wells is about 33 per cent., and the efficiency has increased from 24.3 per cent. to 34.3 per cent.

The well next put in operation is fitted with the same size of pipes as the first. A series of experiments at comparative rates of working was carried out on the second and third wells. The rate of working the second well varied between 14.3 and 32.4 revolutions of the compressor, and the efficiency between 38.6 and 23.4 per cent., the free air used per cubic foot of water raised varying between 3.8 and 7.16 cu. ft. With the third well the rate of working varied between 13.5 and 30.5 r.p.m., and the efficiency between 30.4 and 14.9 per cent., whilst the free air used varied between 5.77 and 11.83 cu. ft. per cubic foot of water raised. There are two possible explanations of the difference in efficiency of these wells, namely: (1), the difference in cross-sectional area of the air- and delivery pipes; and (2), the difference in length of the delivery-pipes. In order to ascertain whether the difference in length was the cause of the low efficiency, a series of experiments was made on the third well, after reducing the length of the delivery-tube by 34 ft. Shortening the delivery-pipe had no beneficial effect on any of the factors governing the efficiency, and the conclusion seems justifiable that the larger area of the pipes in the second well is the principal factor in determining its superiority over the others, and that for wells of this depth a pipe 5 in. in diameter is preferable to one of less diameter.

The diagrams in Fig. 4 have been plotted from the data contained in Tables VI, VII, VIII, and XI (Appendix), and show graphically the efficiency, the volume of free air used per cubic foot of water raised, and the number of gallons of water raised per minute, from each of the wells, at various rates of working. A comparison of the curves for the different wells shows the superiority of the results obtained from the second well. In the diagrams the first well is No. 17, the second No. 38, the third No. 29, the fourth No. 40.

The next series of experiments was made on wells 38 and 39 working together. These experiments were to determine the best conditions of working for raising volumes of water up to 40,000 imp. gal. per hour. Some difficulty was experienced, at the beginning of the experiments, in regulating the supply of air to each well, so that they should yield a proportionate amount of water, in accordance with the results of the previous experiments. This could not be done accurately, as there was no means of measuring the flow from each well separately. The best that could be done was to approximate to the correct amounts by observing the flow from the pipes into the brickwork reservoir. The rate of working varied between 23.7 and 39.8 revolutions of the compressor. At the lower rate of working an efficiency of 36.8 per cent. was attained, the quantity of water raised being 34,200 imp. gal. per hour; at the higher rate of working the efficiency had fallen to 26.7 per cent., the quantity of water raised being 40,300 imp. gal. per hour; thus, by increasing the rate of working by 68 per cent., the quantity of water raised only increased by 12 per cent. Had this series of experiments been made for the purpose of finding the highest efficiency, without regard to the quantity of water raised, the rate of working should have been lower at the commencement, when doubtless the efficiency would have been higher. The results of this series of experiments are represented graphically in Fig. 2. An examination of the curves shows the same fall in efficiency, rise in volume of free air used, and slow rise of the quantity of water raised, with a tendency to decrease after the rate of working reaches a certain point, as was observed in the case of the separate wells.

A series of experiments on working three wells together presented the same difficulty as was experienced when working two wells together, and



Results of Experiments to Determine Efficiency of System Between Minimum and Maximum Rate of Working.

to obtain a proportionate flow from each well the same artifice had to be adopted. The object of the experiments was to find the efficiency when raising about 40,000 imp. gal. of water per hour. A speed of 20 to 22 revolutions was found to yield this quantity of water, and two experiments were made at rates of working so nearly equal that the average would perhaps give the truest values; but it is interesting to note in this case, as in the others, that the highest efficiency was attained at the lower rate of working.

During May, 1904, well 40 having been started, a series of experiments was made with it for comparison with the others. This well differs from the others in the arrangement of the pipes, and in its comparatively greater distance from the compressor. The air-pipe is 2 in. in diameter and passes down the outside of the delivery-pipe. The lower part of the delivery-pipe is 4 in. in diameter, but the upper portion, for 100 ft. from the surface, is 6 in. in diameter. This enlarged pipe was put in with the object of reducing the velocity of the air and water as they approached the surface, in the hope of increasing the efficiency of working. In transmitting the air to this well a fall of 3 lb. per square inch in the pressure was observed, due to the greater distance from the compressor. The results are somewhat better than those from well No. 39, but very much worse than those from well No. 38. They are plotted alongside those for wells Nos. 38 and 39 in Fig. 1, which shows at a glance the relative value of the results in each case. The effect of the alteration in the arrangement of the pipes in this well appears to be beneficial, and it seems probable that had a larger delivery-pipe been used, the results would have been at least as high as those obtained from well No. 38.

The next series of experiments was made in June, 1904; wells Nos. 38, 39 and 40 were worked together, and the air-supply was regulated as before, so that each well gave a proportionate flow. The object of the experiments was to determine the efficiency of the system between a minimum and a maximum rate of working. The minimum rate at which the three wells could be kept running was found to be about 18 revolutions of the compressor. The maximum rate at which it was considered safe to run the compressor was 50 revolutions. The highest efficiency, namely, 41.2 per cent., was again obtained at the lowest rate of working, the free air used being 3.01 cu. ft. per cubic foot of water raised, the water raised being 38,160 imp. gal. per hour. At the highest rate of working the efficiency had fallen to 24.6 per cent., the volume of air used had risen to 5.22 cu. ft. per cubic foot of water raised, and the water delivered was 60,000 gal. per hour. The results are plotted in Fig. 3, the most noteworthy feature of which is the rapid rise of the curve for the quantity of water, at the beginning of the series of experiments, and its very gradual rise towards the end of the series. Had it been found practicable to continue the experiments further, it appears probable that the curve would ultimately have shown a tendency to descend, as in the case of the single wells when the higher rates of working were attained.

In order to ascertain whether the action of the compressed air in an air-lift is similar to that of a piston in a cylinder, or whether the air forms an emulsion with the water, a series of observations and experiments were made on well No. 38, using the small compressor so that a very low rate of working could be attained. The results appeared to show that the air might act in either way, or by a combination of both, depending upon the rate of working. With a very low rate of working, air passed through the well without lifting the water high enough to overflow. As the rate of working was increased, water began to flow intermittently and in cycles. The water would begin to run slowly, the flow gradually increasing until the delivery-pipe was running full; then there would be a rush of air, causing the water to rise in the form of a dense spray, after which the flow would gradually diminish, and finally stop. If the rate of working was kept constant the flow continued in periodic cycles. The series of experiments was commenced at 82 revolutions of the compressor and the rate of working was increased by 20 revolutions per minute at a time, until a speed of 165 revolutions per minute was attained. In experiment 1 the cyclic period was 2 minutes 20 seconds, during 1½ minute of which no water flowed from the well. During the remaining 50 seconds, the flow, commencing slowly, gradually increased until the rush of water and air already referred to took place, after which the flow gradually declined until it ceased altogether. In experiment 2 the cycle occupied 2 minutes, water flowing for 1¼ minute, and no flow taking place during the remaining ¾ minute. In experiment 3 the period was 1¼ minute, the stoppage in the flow lasting only ¼ minute. In experiment 4 the flow was continuous, but varied in volume from time to time. In these four experiments the action of the air appeared to be similar to that of a piston. In experiment 5 the flow of water and air was continuous, and resembled a dense spray, the air appearing to form an emulsion with the water. The efficiency was lowest at the lower rates of working; it reached a maximum in experiment 4; and in experiment 5 it began to fall again.

The results of the foregoing experiments indicate to Mr. Kelly that there is a certain rate of working wells by compressed air at which the maximum efficiency is obtained, and that the best efficiencies are obtained at comparatively low rates of working. Previous experimenters are agreed on the necessity of keeping the velocity of the water in the pipes low, and fix the limits of this velocity at 4 to 8 ft. per second; but from Mr. Kelly's experiments it would appear to be necessary to take into account the combined volume of both air and water in fixing a limit of velocity in the pipes. Where the highest results have been obtained the velocity of the air and water combined has been about 12 ft. per second at the point of entering the delivery-pipe. As the air and water rise to the surface, the velocity continually increases, due to the increasing volume of the expanding air. Having regard to this circumstance, it has been suggested that the delivery-pipe should gradually increase in diameter from the bottom of the well to the surface. An approximation to this arrangement is that adopted in well No. 40. A comparison of the velocity at the top of the delivery-pipe in this well with that in either of the wells Nos. 38 and 39 shows that the velocity at the top was much reduced by increasing the diameter of the delivery-pipe. What effect this had on the efficiency cannot be determined, owing to other disturbing factors. If this point could be investigated under truly comparative conditions, the results no doubt would prove interesting.

THE GUNNISON TUNNEL of the Uncompahgre irrigation project in Colorado, had been driven 14,614 linear feet at the end of July, the progress during that month amounting to 718 ft. Less headway was made in July than in previous months on account of friable material encountered in one heading and on account of a premature explosion, and the amount of water flowing in the other heading. This tunnel is believed to be the longest and largest underground waterway of its kind that has been undertaken. It will have a total length of 30,000 ft., a cross-section of 10.5x11.5 ft., and will be lined with concrete. Its total capacity will be 15,000 cu. ft. per second. For the greater part of its length the tunnel is in solid granite, and in some places is 2,000 ft. below the Vernal Mesa.

A Heavy Center-Bearing Draw Span.

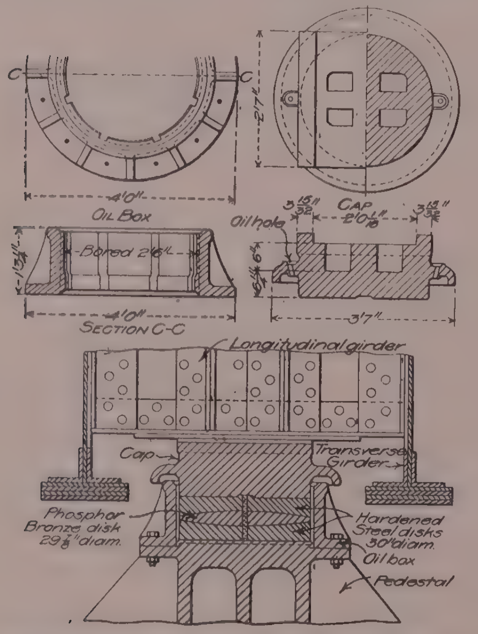
The 270-ft. swing span of the bridge over the Harlem Ship Canal, at Kingsbridge, N. Y., it about 54 ft. wide over all and has two riveted trusses 39 ft. apart on centers and nearly 60 ft deep in the center. It is a double deck structure which carries three tracks of the Rapid Transit Railway system on the upper floor and two electric car tracks, two driveways and two cantilever sidewalks on the lower floor. The total weight of the moving structure is about 3,000,000 lb. and while the bridge is being moved it is supported entirely by the center pivot, except inasmuch as the tipping tendency is met by eight trailer wheels running on a 36-ft. circular track. When under traffic the live load is carried through adjustable wedges to fixed pedestals on the center pier, thus transforming the structure to two continuous spans with clear openings of 104 ft. for navigation. It is believed by the designers that this is the longest and heaviest swing span ever built with center bearing and the simplicity of the design and the satisfactory result which has been secured in its operation lend special interest to the details of construction.

The pivot is subjected to a dead load of 3,000,000 lb. and consists of a set of loose discs supported on a cast-steel pedestal which distributes the load over a circular area of masonry 8 ft. 3 in. in diameter, as indicated by the detailed drawing in the description of the erection of the span which was published in The Engineering Record of Aug. 4. The top of the pedestal casting is faced for bearing on the lower disc 30 in. in diameter which, like the upper disc, is made of hardened steel polished on the concave surface. The lower surface is flat and it is 3¼ in. thick at the center with the upper surface concaved to a radius of 12 ft. 3 in. to engage a phosphor-bronze disc 29⅞ in. in diameter and 3½ in. thick in the center. The upper surface of this disc engages the upper disc, which is like the lower one except that it has a small boss in the center of the upper side to engage the pivot cap. The middle disc is curved to a radius of 12 ft. so that the surfaces are not absolutely in contact with those of the upper and lower discs, or with the oil box. It is also provided with two transverse diametrical oil channels in the upper surface staggered with corresponding ones in the lower surface. All three discs are held together by a 1-in. center vertical screw and are enclosed in the cast-steel, oil box, 15¼ in. deep, flange-bolted to the top of the pedestal and provided with eight vertical recesses in the inner periphery which serve as reservoirs and distribution channels for oil poured in through the two holes in the cap, to keep the discs flooded. The cap is a steel casting engaging the upper disc and provided with a circular flange covering the top edge of the oil box.

The method of transferring the weight of the span to the pivot is different from the ordinary system of double center girders resting on the pivot from which the span is generally suspended by adjustable vertical bolts which, after the preliminary setting, are generally never readjusted. It was therefore thought desirable to provide a support without adjustment and it was accomplished as shown in the general plan and cross-section with a very simple arrangement and less girders than are generally required for such a purpose. The trusses are connected at the center panel point by a pair of heavy transverse girders with their top flanges flush with the bottom flanges of the floor beams. These girders are connected to the adjacent floor-beams by four longitudinal girders and by four inclined struts connecting the ends of both sets of girders. The trailing wheels, carried on the longitudinal girders, are connected with the transverse girders by diagonal struts. The transverse girders are web-connected together on the center line of the bridge by a longitudinal plate girder, 6 ft. long, with a 1½-in. web, 108 in. deep, and by four 8 x 8 x 1-in. flange angles and four pairs of heavy vertical web-stiffener angles at the center. The span is symmetrical longitudinally, and transversely about this girder which is concentric with the pivot and being supported at its center carries the span, which is balanced about this point. Its lower flange takes bearing on the upper side of the pivot cap, where it is guided by longitudinal ribs integral with the latter and is held in the opposite directions by transverse bars riveted to the girder flange inside of the cap, thus maintaining it securely in place, although it would otherwise be immovable horizontally on account of the friction.

At each end of the main transverse girders a cast-steel pedestal is attached to the lower flanges and is provided with wedges similar to those usually employed for lifting the ends of draw bridges. These wedges are driven in when the span is closed and then transfer the live load directly to the pier, relieving the pivot of all live load pressure and together sustaining a maximum live load of 2,500,000 lb. The 3,000,000 lb. load on the pivot produces a maximum pressure of 4,250 lb. per square inch on the disc, an amount which would have been inadmissible until the recent improvements in the manufacture of bronze have developed greatly increased hardness and bearing strength.

The bridge has now been completed and in operation for several weeks and the pivot gives excellent satisfaction in service. The bridge is operated smoothly and easily in four minutes by two 60-h.-p. General Electric motors, while the 1,218,000-lb. single-deck structure which it replaced required about 15 min. to operate it with steam power. The bridge was designed by the engineering department of the Rapid Transit Railway Commission, Mr. George S. Rice, chief engineer; Mr. Alfred Craven, deputy chief engineer; Mr. Sverre Dahm, general inspector of designs. The American Bridge Co. was the contractor for the superstructure and operating mechanism and the Terry & Tench Co. was the contractor for its erection.



Support of Center Girder on Pivot Cap.

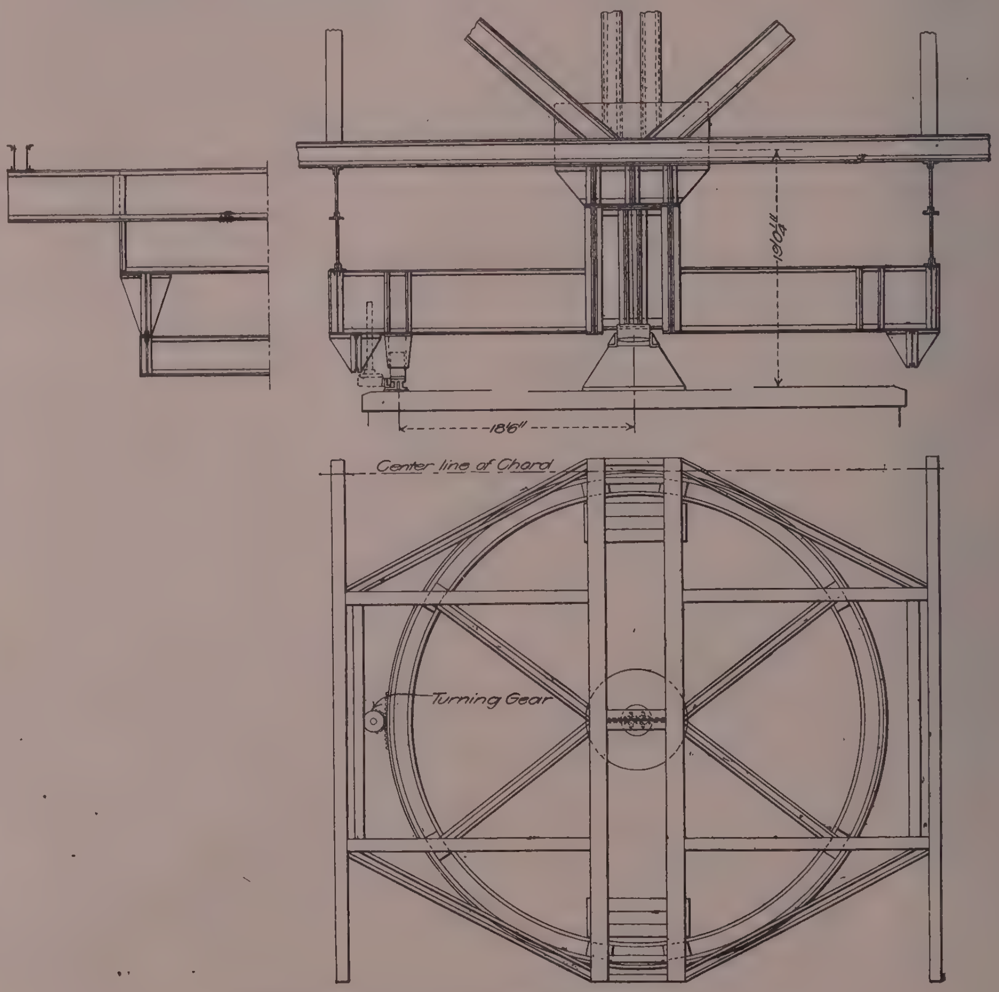
Lowering a 30-ft. Brick-Lined Well from a depth of 18 ft. to 40 ft. was recently accomplished successfully by the Marysville, Kansas, water works in order to increase the water supply. It was necessary to pump 750 gal. per min. from the well while excavating.

Sludge Treatment in Relation to Sewage Disposal.

A paper read at the annual meeting of the Association of Municipal and County Engineers by John D. Watson.

Considering the great changes that have taken place in the views of sanitarians in ten short years, it behooves us to approach the question of sewage purification in a less dogmatic manner than some of us have been in the habit of doing, and to express our conclusions and the results of our experience without presuming to say that because a certain course is suitable at one place it must necessarily be suitable at another, that because a scheme is found to answer well in the North it must be equally applicable to the South.

Had sewage always been composed of waste from dwelling houses there might have been some excuse for this attitude, but where domestic sewage is mingled with trades waste which contains acids, sulpho-cyanides, pickling liquors, brewery wastes, wool washings, etc., it ought to be obvious to the least intelligent that different methods of treatment must exist, and that any attempt to deal with the question in a stereotyped manner will only put back the hands of the clock.



Center girders and Bracing over Pivot Pier.

In order to guard against misapprehension therefore it should be clearly understood that in the remarks and recommendations that follow the author is dealing with a sewage having the characteristics shown in Table I.

The work of sewage purification may be said to be divided into the following sections:

(*a*) Arresting and disposing of solids, like road grit, coal slack, rags, etc., which in the bulk forms sludge containing 50 per cent. water.

(*b*) Precipitating organic sludge containing 90 per cent. water, converting it into an inodorous substance before disposing of it on land or otherwise.

(*c*) Aëration and nitrification of liquid sewage per se and its subsequent discharge into a river in a state that will increase in purity as it flows down stream.

This paper will deal briefly with phases of the first two sections of the subjects, viz., (*a*) and (*b*), and this appears to divide itself into three main considerations: (1) The means of effectually arresting solids, (2) removing them quickly, and (3) disposing of them without nuisance.

Table I.

Average Daily Analyses of Birmingham Crude Sewage.

Figures are for 1903-05 inclusive, and are in parts per 100,000.

Dissolved solids	119.3
Suspended solid:	
Total	74.3
Organic	44.9
Ammonia:	
Free and saline	4.05
Albuminoid	1.57
Chlorine	20.2
Nitrates and nitrites, as nitrogen	0.92
Oxygen absorbed:	
Unfiltered	27.56
Filtered	15.79

It is possible to treat some sewages in their crude state without abstracting the grosser solids, but where there is a large quantity collected from a large area—particularly if trades waste constitutes a prominent feature of the sewage—it may be taken that means for abstracting the solids must form an essential feature of every purification scheme.

The Arrest of Solids.—The methods adopted for abstracting detritus from sewage are numerous, and must necessarily vary with circumstances, but it may be assumed that a tank or grit chamber plus some mechanical arrangement for cleaning the same is absolutely indispensable. The size of this chamber in its relation to the volume of sewage to be dealt with is a subject that requires the closest attention before any special design is decided on, and with all the information and experience in the possession of engineers few feel able to lay down an unvarying standard of construction either as regards form or capacity.

After not a few trials—and a few failures—the author has come to the conclusion that the grit chamber should be, where possible, in the form of an inverted pyramid, and that it should be fitted with a mechanical arrangement for dredging out the solids capable of being set in motion at the discretion of the attendant.

In principle, an inverted cone would equally serve the purpose of arresting suspended matter, but the apex of a cone is a most awkward shape for fixing anything in the nature of a dredger, and for this reason also a true pyramidal form is not exactly ideal. Perhaps the best shape of all for a detritus chamber is an elongated pyramid, inverted, of course.

It frequently happens, however, that one has to deal with works that have been constructed for many years, and in such cases the shallow rectangular tank is usually found to exist. This was so at Birmingham, where two tanks of this form, capable of holding 1,200,000 gal. each, were constructed in 1859 by the late Mr. Thomas Hawksley. Until 1901 these tanks, and others of a similar kind constructed subsequently, were used as precipitation tanks. They were divided into three bays or sections, and milk of lime in the proportion of 12 grains per gallon was mixed with the sewage before it entered the tanks for the purpose of neutralizing acids and assisting in the precipitation of suspended solids.

As will be readily understood, the road grit was for the most part arrested in the first bay of the tank, the second and third bays arresting sludge containing a larger proportion of liquid and organic matter. When these tanks were first used the greater part of the cleansing operations was done by manual labor. Subsequently fixed stationary dredgers were erected for the purpose of lifting into wooden carriers sludge containing 90 per cent. of water. These dredgers were found to be unable to cope with the work required of them, and Tangyes' direct-acting pumps were employed to assist them, and more recently a traveling crane, fitted with a Priestman grab, was employed to lift the grit out of the first bay of each tank. This process was continued for many years, but the amount of sludge arrested was so enormous, amounting to about 260,000 cu. yd. per annum, and the difficulty of dealing with that sludge when it was arrested so great, that it was evident some drastic improvements were required if the difficulties were to be grappled with successfully.

The chief difficulty did not lie in the want of ability to arrest the sludge, for the tanks did this part of the work admirably, size in relation to the volume of sewage being an important element in the case.

Tank Flow and Tankage.—The average rate of flow of sewage through the tanks was, up till 1902, 2.05 ft. per minute, and the average disposition of the solids due to a combination of mechanical and chemical action 75.9 per cent., and to mechanical action alone 56.4 per cent. of the total suspended matter in the sewage.

Since the number of tanks was increased the rate of flow has been reduced to 1.20 ft. per minute, and the deposition without the aid of chemicals increased to 59 per cent.

At Hanover a set of experiments was made a fews years ago on tanks 246 ft. long, with the view of ascertaining the most advantageous rate of flow. With a velocity of 9.44 in. per minute, 62.7 per cent. of deposition was obtained; with a velocity of 14.7 in. per minute 61.7 per cent. was obtained; with a velocity of 17.43 in. per minute 57.3 per cent. was obtained.

The nature of the sewage dealt with has a considerable influence on the deposition of sludge, that is, at Birmingham there is a large proportion of macadamized roads and streets, with the result that organic solids, like road grit, precipitate quickly and assist the deposition of lighter substances in their downward course.

Probably the chief difficulty at Birmingham lay in the inability of the executive by reason of inferior plant to empty the tanks quickly. During the liming process the roughing tanks were not emptied more frequently than once per month, and the process of emptying occupied a week. The finishing tanks were emptied about once every three months.

In June, 1902, both roughing and finishing tanks were increased in number, adding nearly 50 per cent. to the tankage. While this addition had the effect of improving the work of arresting sludge, the amount arrested going up by 23 per cent., it also had the effect of making more evident the weakness of the plant available for removing sludge from the tanks. In April, 1903, however, new pumping plant for raising sludge was brought into use. By means of it each roughing tank could be emptied once a week, instead of once a month. This virtually meant an increase in effective tank capacity, and still more effective sedimentation was obtained, the sludge removed rising during the following year by 44 per cent., while the suspended solids in the roughing tank effluent fell from 29 to 17 parts per 100,000. The suspended solids in the septic tank effluent fell during the same period from 23.2 to 13.1 parts per 100,000.

A curious phenomenon was now observed in the septic tanks. The quantity of suspended solids was much reduced in the liquor passing into these tanks and yet they began to silt up, while their activity as septic tanks was much reduced. This was accounted for as follows: The organisms in our crude sewage are remarkably few in number, such observations as have been made giving an average of 500,000 or so per cubic centimeter. The long sojourn of much of the sludge in the roughing tank under the old method of emptying (once a month) resulted in much putrefaction with consequent increase in the organic life in the tank liquor. The liquor, therefore, being passed to the septic tanks was in an incipient septic state under the old conditions. Under the quicker methods of sludge removal this incubating ground for the microbes was removed, while the organisms already in the septic tanks were deprived of much of the food (sludge) they formerly received.

For this paper it is unnecessary to describe the nature of the new pumping plant beyond stating that in conjunction with increased power to lift and force sludge an underground main was laid for its conveyance down the valley. Branches were fixed to this main, which is about 3 miles long, at every 200 yd., and to these branches are attached, when required, portable steel pipes, to convey sludge to any particular field.

This addition to the plant effected a great change in the work of sludge disposal, and at first glance it appeared to have defeated its own object, inasmuch as the more efficient the means of dealing quickly with the sludge the more there was to deal with. One day was now sufficient to empty one tank, and as there were but five large precipitation tanks to empty (the smaller tanks all having been converted into septic tanks) the whole of the tank cleaning operations were confined to five days of the week, beginning with Monday and ending with Friday.

The Sludge Nuisance.—Having now got to the stage when it was practicable to remove, for a sum of 3d. per cubic yard 80 per cent. of the suspended solids in five days of ten hours per day (without reckoning the time required to pump the supernatant water), and having got what was of even more importance, the ability to remove sludge from the tanks before it was sufficiently decomposed to give rise to nuisance, a great advance on former conditions had been achieved, but there was still something to do, viz., to abate the nuisance arising from the exposure of semi-putrid sludge in the fields.

Lord Bramwell's Commission stated that getting rid of the sludge was the crux of the sewage disposal question, and to be obliged to get rid of 800 tons of it per day without creating a nuisance, or what is less easy, without giving rise to complaints, is a matter of the greatest difficulty. Where it can be shipped out to sea whenever it is taken out of the tank the difficulty is almost solely an economical one, but where it has to be pressed into cake or got rid of on land, the difficulties are both hygienic and economic.

In the case of a large farm the usual practice has been to spread the sludge in great shallow lagoons over an area of ground, and allow it to dry sufficiently to permit of its being dug into the ground as a gardener digs in manure, a practice good in itself, but one which frequently gives rise to nuisance, not only when the work is being done, but afterwards, when the sludge is disturbed by the plough. Then it frequently happened that the area under sludge at one time was so great that it was impossible to cope with the nuisance.

This method of disposal was improved on and the risk of nuisance minimized to a very large extent by cutting trenches 3 ft. wide and 18 in. deep, and pouring the sludge into these trenches, covering them over with earth as soon as practicable, but even this method contained elements of nuisance which were most difficult to contend with during certain seasons of the year, especially where the amount of sludge to be got rid of occasionally amounted to three-quarters of a mile of trenches per day.

Experiments in Sludge Deodorization.—Many experiments were tried to get rid of the objectionable odor of the sludge, some of which may be enumerated, viz., soot, a combination of lime and soot, spraying paraffin on the surface, a substance advertised by the name of Chlorous, a solution of bleaching powder containing hypochlorite of calcium, none of which could be regarded as efficient, and all of which were costly.

The most efficient experiment in the direction of deodorizing the sludge by artificial means was the application of a solution of bleaching powder containing hypochlorite of calcium, having 36.35 per cent. or available chlorine as it was pumped from the tanks into the sludge main in the proportion of 35 lb. by weight of powder to 24,000 gal. of sludge. This experiment when tried in the laboratory was found to be a great success, converting the nauseous sludge into a pleasing smelling substance, but when the solution was added to the sludge at the outfall works, where it was all mixed up before it was drawn through the pumps and churned up in the pumping main, the results were not so good, and the greatest possible difference between the laboratory experiment and the actual working was made manifest.

Since lime treatment was abandoned and septic treatment inaugurated in 1901, the whole of the land between Saltley and Curdworth in the Valley of the Tame has been subjected to irrigation by septic sewage, and on several occasions when the amount of suspended matter in the septic sewage was more than usually great, the author was much struck by the absence of objectionable odor. On looking at the land grips, which in places were covered with a thick black paste deposited only the day before, or it may have been on the day of observation, he was amazed to find that the sludge had practically no odor at all. This led to further observations which all tended to confirm the fact that septic sludge, or sludge which formed the residuum of the septic tank process, was devoid of fetid odor, and indeed that it required a very acute sense of smell to detect odor at all.

Septic Tank Residuum.—The outcome of this observation brought about the emptying of a septic tank which had been in constant use for three or four years, with the view of ascertaining whether the sludge in bulk would have the same inoffensive characteristics. The experience gained was perfectly satisfactory, the residuum in bulk having been proved to be as inodorous as the thin layers which were first observed. This experiment was repeated from time to time always with the same results, and in January of 1904 all the septic tanks were emptied and the residuum pumped on to an adjacent field about 8 acres in area, where it was allowed to accumulate, until in its liquid state it measured on the average 18 in. deep. This sludge was allowed to lie for seven months without being touched, and during that time it continued to retain without variation its inodorous character, whether the temperature was high or low. Father Time rendered his help, and the depth of sludge was reduced both by evaporation and absorption until the quantity to be dug into the ground did not exceed from 6 to 9 in. in depth. This was then ploughed into the ground by steam plough, allowed to lie for a month or two until the season for sowing rye came round, when a crop of rye was sown. The crop raised, however, was not satisfactory, but this was not unlooked for, as the residuum of the septic tank process contained such a large amount of metallic salts that when it was used by itself for the purpose of ascertaining whether common garden seeds could be made to germinate in it, it was found that the seed (perilla) would not spring into life. The residiuum was then mixed with an equal proportion of black earth, and again sown with similar seeds, when about one-third of them germinated, but did not mature. Not until the proportion of residuum was reduced to one-fourth of the black earth did the perilla show signs of a moderately healthy life. It was therefore quite clear that it was a mistake to run so much sludge on the surface of the land at one time, and in subsequent working not more than half the amount of sludge has been put on the surface of the ground, with far more satisfactory results; besides, the shallower depth dries much more quickly, and the steam plough is better able to cover it over with good soil. In the first trial a mistake was made in the crop chosen, viz., rye. The roots of that cereal penetrated too far into the ground, going in fact into the residuum which had been purposely ploughed under the surface; in the subsequent trials Italian rye grass was sown with satisfactory results.

An anaylsis of this residuum was made by our chemist, and is as follows. The inorganic matter contains:

	Per cent.
Siliceous matter (sand, etc.)	20.00
Oxide of copper	1.71
Oxides of rion and aluminum	20.00
Oxides of zinc, manganese and nickel (about)	5.00
Lime and magnesia (about)	7.00
Oxides of phosphorous and alkalies (about)	2.00
Organic matter in dry sludge	44.67
Nitrogen	2.47

Tank Treatment Experiments.—While these observations were being made in the early part of 1904, certain experiments were being made on tank treatment. As I have already stated, between April, 1903, and April, 1904, the roughing tanks were being cleaned out much more frequently, and the result was to denude the septic tanks of a considerable portion of the liquid organic sludge which formerly went to feed them. It was established that although the efficiency of the roughing tanks for sedimentation had been increased 44 per cent., the septic tanks gave unmistakable evidence of becoming less active. Instructions were then given that a considerable portion of this lighter sludge should be pumped from the roughing tanks into the septic tank inlet channels and made to mix with the sewage passing from the roughing tanks to the septic tanks. This liquid sludge was greatly increased until a volume equal to 30 per cent. of the total sludge deposited was passed into the septic tanks daily. The result was almost an immediate restoration of active fermentation in the septic tanks.

When the importance of this discovery was realized it became necessary to reconsider the position of the septic tanks in relation to purification of sewage, and the first question that occurred was how much sludge does the septic tank get rid of. This the author answered in 1902 in giving evidence before the Royal Commission on Sewage Disposal as 10 per cent.; a similar answer was given to the same commission in 1905. Since then (while the most active fermentation has been maintained) observations go to show that little more than 10 per cent. of the total solids entering the septic tanks is converted into gas.

This figure shows a considerable discrepancy with those given by Dr. Kinnicutt, of Massachusetts, in a recent paper, where he pointed out that Exeter, Leeds, Manchester in this country, and at Worcester, Mass., agreed in stating that about 25 per cent. of the organic sludge was gasified by bacterial action in a septic tank. The quotation is from memory, and is, of course, subject to correction, but the disparity between Birmingham and the other places is so great that it calls for some observation, and it should be ascertained whether all the estimations were carried on for a long or short time, and whether or not they were confined to comparatively small volumes of sewage. In Birmingham the estimation of 10 per cent. is the result of nearly five years' observation of an installation dealing with 30,000,000 gal. of sewage per day.

Having arrived at the conclusion, notwithstanding the experience of others with other sewages, that the Birmingham sludge could not be gasified in a septic tank to a much greater extent than 10 per cent. of its bulk, and having discovered that the sludge or residuum formed in the bottom of the septic tanks is without objectionable smell, the author came to the conclusion that the real solution of the sludge difficulty was probably to be found in utilizing the septic tanks as manufactories of septic sludge, if that phrase may be used to denote the residuum of the fermentation process, more particularly as the only sacrifice involved in the change was the more frequent emptying of the tanks. Instead of cleaning once every two years as formerly, last year they were emptied on an average once per seven weeks.

No sooner had this resolution been taken than instructions were given to carry it into effect. (1) By connecting all the suction pipes in the septic tanks—they formerly terminated at a temporary pumping station in the middle of the tank area—to the engine and pump house. (2) By converting as much of the precipitation or roughing tanks into septic tanks as could be spared, bearing in mind the necessity for retaining detritus tanks. The first work calls for no comment.

Conversion to Septic Tanks.—To determine what proportion of the roughing tank should be retained to act as a detritus chamber was not without difficulty. As, however, there were several division walls in the existing tank, it was possible to act cautiously by contracting the size of the existing chamber from time to time. Ultimately it was determined that the most suitable capacity of the detritus chamber should be 20,000 cu. ft. per tank, representing 3,333 cu. ft. per 1,000,000 gal. treated.

The system, although incomplete, has now been in operation for more than a year, and the following brief description of the working will help to elucidate the method adopted.

The sewage enters the roughing tanks through the detritus chambers, where it travels at the rate of 1.20 ft. per minute, leaving behind in the detritus chamber the larger proportion of its inorganic matter, that is, fine sand, gravel, particles of coal, cinders, wood, leather, animal hairs, etc., and in the septic tank, which is reached by passing over a submerged weir and under a scum board, the finer particles of suspended matter that escape the detritus chamber, and which include a larger proportion of organic matter, such as animal fat, paper, fragments of food, vegetables, and much fæcal matter, are deposited.

Only a floating scum board separates what looks like a placid liquid charged with fine sand and a water resembling black ink, fermenting so actively that it may be heard as well as seen, a condition which was attained only after the greatest difficulty owing to the paucity of microorganisms in the sewage.

The large septic tank maintains its activity, notwithstanding the fact that a scum is positively discouraged; the object of this is to allow the suspended matter which is inflated by the gases rising from the tank liquor to pass freely to the septic tanks proper.

The detritus tank is emptied once a week. The supernatant water is pumped off and the slimy part of the sludge is pumped into the channels which feed the twenty septic tanks with sewage, thus mixing sludge and sewage together. That part of the sludge which is too highly charged with sand and grit is sent direct to the sludge trenches, and the still more solid stuff is lifted out by steam grab and buried at once.

Tanks 1, 2 and 3 are treated in precisely the same way, only the detritus tanks being much larger there is more work to be done.

TABLE II.

TABLE BASED ON FACTS OBTAINED IN WORKING TANKS FOR YEAR ENDED MAY, 1906.

Cu. yd.		£	s.	d.
19,277	=quantity of sludge (90 per cent water) pumped to trenches from each of tanks Nos. 1, 2 and 3 per annum.			
7,284	=quantity of sludge (90 per cent water) pumped to trenches from each of tanks Nos. 4 and 5 per annum.			
11,993	difference × 3 (quantity pumped to trenches that could have been obviated)=35.979 cu. yd. at 2.85d..	£427	5	0
36,202	=quantity of sludge (90 per cent. water) pumped into septic tanks from each of tanks Nos. 1, 2 and 3 per annum.			
15,016	=quantity of sludge (90 per cent water) pumped into septic tanks from each of tanks Nos. 4 and 5 per annum.			
21,186	difference × 3 (quantity pumped to the septic tanks that could have been obviated)=63,558, cu. yd. at, say, ½d. per cu. yd.	130	0	0
	In the same way it is calculated that it would be unnecessary to pump 29,573,232 gal. supernatant water per annum, if detritus chambers of tanks 1, 2 and 3 corresponded in size with tanks 4 and 5; 29,573,232 gal. at, say. ½d. per 1,000 gal.	60	0	0
		617	5	0
	From this must be deducted the extra amount of sludge to be removed from the septic tanks—90 per cent of 35,979 cu. yd.=32,381 cu. yd. at 1d. per cu. yd.	134	18	0
	Probable saving of working costs if tanks 1, 2 and 3 coincided in size with tanks 4 and 5	£482	7	0

The accompanying Table II shows the probable effects of altering tanks Nos. 1, 2 and 3 to correspond with Nos. 4 and 5, and the leading feature to be observed is that there would be an economic saving on every process, and an aggregate saving of £482 per annum, less cost of constructive work necessary to effect the change.

In 1900, the last year when lime was used, the cost of sludge disposal was 7.2d. per cubic yard. For the year 1904, after the new plant was introduced, the actual cost was 2.968d. per cubic yard, and since it has become practicable to use the inodorous sludge for the purpose of filling up depressions in the various fields to which it is applied (thus obviating the cutting of trenches, etc.), the cost has been lowered to 1.85d. per cubic yard. All the figures are comparative, and the lowest will not be reached until tanks Nos. 1, 2 and 3 are made to proximate to their fellows.

A Rapid General Method for the Calculation of Reinforced Concrete Sections

By Richard T. Dana.

To the engineer who has to solve problems of design in reinforced concrete and who to guide his practice makes a comparison of the data bearing on the subject, the choice of formulas and methods is a difficult and often a most perplexing question. If he desire a good formula for the moment of resistance of a proposed beam, for example, so far from being unable to find one, he is at the outset confronted with a large number of good formulas, derived by an equally large number of good concrete men, each of whom represents his units by his own particular set of algebraic characters, and most of whom have certain (and different) assumed constants. The result is that the designer has before him a large number of methods the caligraphy of which is so diverse that, while based on not more than two or three original theories of flexure, no two of the formulas look alike.

Nearly every set of formulas has to be used in connection with various constants derived or accepted by the sponsor of the formula and which ought to fit the case in hand, but rarely do. The writer has felt the need in his office of a method of analysis, or set of formulas, that would be general in form, rapid of application, and in which the various constants dependent upon the peculiarities of the materials employed could be readily inserted by the designer without loss of time. Moreover, the choice of a formula will mean a good deal in the economy of the design, there being a large variation in the results by different formulas, owing to the difference in the assumed constants.

The design of reinforced-concrete structures ought to be on the same practical plane as that of steel bridges, subject to the peculiarities of the materials employed, but not subject to variations in the theory of flexure. Having suffered a good deal from the difficulty of having to choose a formula for a particular problem, the writer has worked out the formulas and diagrams here presented. They have done good service in his own office, and it is believed that their use will result in a considerable saving of time for others. The fundamental theory and assumptions are based on those of Coignet, Tedesco and Maurel, and were published as early as 1894. They may be found in "Resistance des Materiaux Appliquée au Beton et au Ciment Arme," by Tedesco & Maurel.

Assumptions.—(1) The stress-strain curve for concrete in compression in the presence of steel reinforcement up to the safe load is a right line. (2) The neutral axis of any homogeneous section passes through the center of gravity of the section. (3) The deflection of any fibre is small compared to its length. (4) Sections plane before flexure remain plane after flexure. (5) The elastic limit of the material is not exceeded. (6) Concrete is inefficient in tension. (7) A heterogeneous section of concrete and steel can be transformed into a homogeneous concrete one by multiplying the area of the steel by the ratio of the coefficients of elasticity of the two materials. (8) There is a sufficiently perfect bond between the steel and concrete to make them act together. The first five of these assumptions are those common to the generally accepted theory of flexure for steel beams. The sixth is in accordance with standard practice in this country, and represents conservative practice. The seventh may be demonstrated in the following manner:

A beam in elevation, sectioned at a point distant L from the right support, and deflecting under vertical loading, is shown in the accompanying Diagram No. 1. Assume the following algebraic values of the nomenclature in that diagram:

v' = distance from center of metal in tension to neutral axis.
d' = distance from extreme compression fibre to neutral axis.
b = width of beam, or width of vertical leg of T-beam.
b' = width of horizontal leg of T-beam.
S = area of steel in beam or slab.
C = area of concrete in compression.
E_s = coefficient of elasticity of steel.

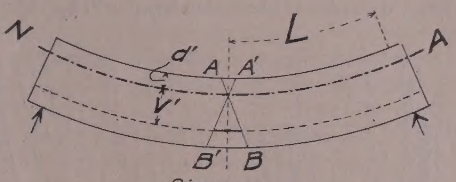


Diagram 1

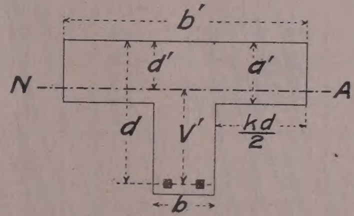


Diagram 2

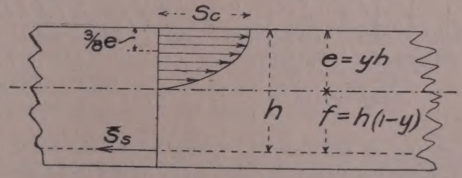


Diagram 3

E_c = coefficient of elasticity of concrete in compression at working stress.
$n = E_s/E_c$.
I = moment of inertia of section reduced to concrete basis.
M = moment of resistance of section in inch-pounds.
R_s = mean unit working stress of steel in pounds per square inch.
R_c = maximum unit working stress of concrete in compression.
r = ratio of area of steel to that of concrete above the steel in rectangular beam.
d = distance top of beam to center line of steel in tension.
w = weight per linear inch including the weight of the beam.
l = length of beam in inches center to center of supports.
$m = R_s/R_c$.

In Diagram No. 1 let the elongation of the steel fibre under investigation be λ in the length L under the loading. Let the stress in this steel fibre be represented by R_s', and the stress in the adjacent concrete fibre be represented by R_k'. Also let the area of the concrete fibre adjacent to the steel be represented by K.

Then, R_s'/S = unit stress in steel fibre, and R_k'/K = unit stress in concrete fibre. Also λ/L = unit strain in each fibre.

Now, since coefficient of elasticity equals unit stress divided by unit strain,

$$E_s = \frac{R_s'/S}{\lambda/L} = \frac{R_s'L}{S\lambda} \qquad (1)$$

Also,

$$E_c = \frac{R_k'L}{K\lambda} \qquad (2)$$

Dividing (1) by (2), we have

$$E_s/E_c = n \qquad (3)$$

Now let $R_s' = R'_k$.

Then $K = nS$, which is the algebraic expression of assumption 6.

Assumption 7 seems to be justified by experience, at least where bars of varying section are used.

The analysis of special shapes can generally be accomplished most readily by finding the neutral axis by trial and obtaining the moment of inertia by direct computation. Rectangular and T-beams can be solved by means of the following formulas and diagrams:

Rectangular Beams.—Considering no concrete in tension and no steel in compression.

From Diagram No. 2, neglecting the concrete below the neutral axis and tabulating, we have for equilibrium,

$$d - d' = \frac{bdd' - \dfrac{bd'^2}{2}}{bd' + nS} \qquad (4)$$

Clearing of fractions,

$$bdd' - bd'^2 + nSd - nSd' = bdd' - \frac{bd'^2}{2} \text{ and}$$

$$\frac{bd'^2}{2} + nSd' = nSd. \qquad (5)$$

Solving this quadratic:

$$d' = -\frac{nS}{b} + \sqrt{\frac{nS}{b}\left(2d + \frac{nS}{b}\right)} \qquad (6)$$

Now, by inspection, $I = \dfrac{bd'^3}{3} + nSv'^2 \qquad (7)$

where $v' = d - d'$, and neglecting the moment of inertia of the steel about its own axis.

Now, from the general theory of flexure $M = \dfrac{RI}{v}$, where R is the unit stress in a fibre distant v from the neutral axis of any section whose moment of inertia is I.

$$\text{Therefore, } M = \frac{R_cI}{d'} \quad \text{or} \quad M = \frac{R_sI}{nv'} \qquad (8)$$

$$\text{Whence we have } R_c = \frac{Md'}{I} \qquad (9)$$

$$\text{and } R_s = n\frac{Mv'}{I} \qquad (10)$$

Equations 6, 7, 9 and 10 are all that are strictly necessary for the investigation of beams of this type, of which the slab is a special case. They are general, and on them are based those which follow.

Now divide equation 10 by equation 9, and we have:

$$\frac{R_s}{R_c} = \frac{nv'}{d'} = \frac{n(d-d')}{d'} = m \qquad (11)$$

Clearing of fractions, $nd - nd' = md'$, and

$$\frac{d'}{d} = \frac{n}{m+n} \qquad (12)$$

Take $d' = dx$. Then $x = \frac{n}{n+m}$ (13)

From equations 8 and 7,

$$M = \frac{R_sI}{nv'} = \frac{R_sbd'^3}{3nv'} + \frac{R_snSv'^2}{nv'} =$$

$$\frac{R_sbd'^3 + 3R_snSv'^2}{3nv'} \qquad (14)$$

From equation 5, $bd'^2 = 2nS(d-d')$.

$$\text{Therefore,} \quad M = R_sS\,\frac{(2d' + 3d - 3d')}{3} =$$

$$R_sS\left(d - \frac{d'}{3}\right) \qquad (15)$$

From (12):

$$d - \frac{d'}{3} = d\left(1 - \frac{1}{3}\,\frac{n}{m+n}\right) \qquad (16)$$

and

$$M = R_sSd\left(1 - \frac{1}{3}\,\frac{n}{m+n}\right) \qquad (17)$$

Call the quantity in the parenthesis Q, then

$$M = QR_sSd, \qquad (18)$$

which is a general form of equation proposed by several writers.

$$\text{Now, } r = \frac{S}{bd}, \text{ whence } \frac{S}{b} = rd. \qquad (19)$$

Taking this value in equation 6, and placing d' equal to dx, we have

$$dx = -nrd + \sqrt{n^2r^2d^2 + \frac{2nr^2d^2}{r}}, \text{ and}$$

$$x = r\left(-n + \sqrt{n^2 + \frac{2n}{r}}\right) \qquad (20)$$

Combining equations 20 and 13 we have:

$$\frac{n}{m+n} + rn = r\sqrt{n^2 + \frac{2n}{r}} \qquad (21)$$

Squaring, and solving for r,

$$r = \frac{n}{2m(m+n)} \qquad (22)$$

Q is expressed graphically in Diagram 4; Equation 20 is expressed graphically in Diagram 5; Equation 22 is expressed graphically in Diagram 6

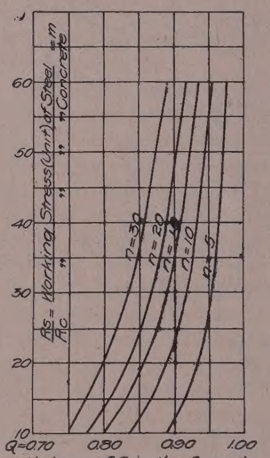
Applications.—The manner of using these formulas and diagrams can be shown best by an example:—Suppose we have given $M = 975{,}000$ in.-lb.; $R_s = 15{,}000$ lb. per square inch; $R_c = 625$ lb. per square inch; $n = 20$; $m = 24$; $d = 20$ inches. Then, from Diagram 6, r is 0.0096; from Diagram 5, x is 0.455; from Diagram 4 Q is 0.85. M is dQR_sS, therefore, S is M/dQR_s, or 3.82 square inches. Also b is S/rd, or 19.9 inches. The location of the neutral axis, if it be desired, is xd inches below the top of beam, or in this case 9.1 inches.

It will be noted that all these figures can be taken directly from the diagrams with the aid of a slide rule. The method affords small opportunity for errors, and the calculations can be safely checked by an ordinary draftsman, this being a large element of office economy.

A comparison of the working formulas gives some interesting results:—From equation 22, r depends only on m and n. If, then, m and n are constant, which is likely to be the case for all of the beams and slabs in any one piece of work, r will be fixed; Q and x will also be fixed. In making estimates, the total steel for direct tension can be quickly obtained by multiplying the concrete in beams and slabs by the factor r, giving the cubic feet of steel in the work. Where T-beams are figured, allowance must be made for the concrete left out.

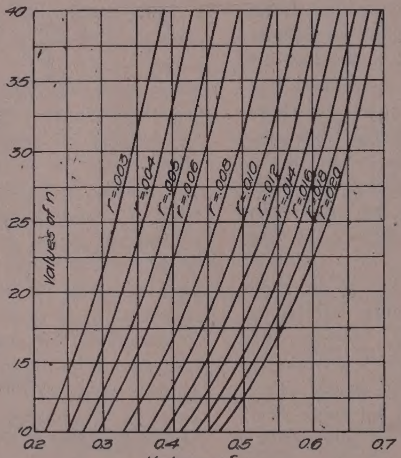
Since $S/bd = r$, a beam 20 in. broad and 10 in. deep will require the same amount of steel to develop the working stresses in the two materials as a beam 10 in. broad and 20 in. deep. Depth here means distance from top of beam to center of steel.

From equation 18 the latter beam will be twice as strong as the former, costing about the same.



Values of Q in the formula $Q = 1 - \frac{1}{3(\frac{m}{n}+1)} = 1 - \frac{x}{3}$

Diagram 4



Values of x

$x = r\left(-n + \sqrt{\frac{2n}{r} + n^2}\right)$

Diagram 5

T-Beams.—There is necessarily much economy in the use of T-beams, of which there are three cases, namely: First, where the neutral axis is above the lower edge of the slab; second, where the neutral axis coincides with the lower edge of the slab; and third, where the neutral axis is entirely below the slab. The second case may be considered a special condition of the first.

Many T-beams are made by pouring and tamping the vertical leg first, and afterward placing the slabs. Now, even with stirrups the bond is not perfect. Therefore it has been the writer's practice to so proportion the T-beam as to have the plane of stress-reversal, or the neutral axis, within the slab.

There are a number of reasons to recommend this practice, one of which is that it affords an excellent specification against putting too large a proportion of the beam into the head, and thus overloading the concrete in shear. If in designing case two be taken as the limiting condition, and case three be barred, the design will be insured against the contingency of too thin a slab for the weight of beam.

For cases one and two, the same formulas, methods and diagrams apply as in the rectangular beams, substituting b' for b and making b just enough to transmit the web stresses.

Comparisons of Other Methods.—The formulas derived above afford a convenient means of comparing other formulas and methods, particularly in their economic bearing.

First, consider the parabolic method, which is based on the assumption that the stress-strain curve is a parabola with its focus on a level with the top of beam for rectangular beams with steel in tension only, the stresses being carried to the ultimate point. This method has been widely used in this country, and deserves more than passing mention. The assumptions on which it is based are as follows: (1) The stress-strain curve up to the breaking point is a barabola. (2) Sections plane before flexure remain plane after flexure. (3) There is a perfect bond between the concrete and steel.

Now, taking a new set of algebraic characters, in order not to confuse the methods, from Diagram 3 and from the book by Messrs. Taylor & Thompson, page 567:

E_c = coefficient of elasticity of concrete, as before.

E_s = coefficient of elasticity of steel as before.

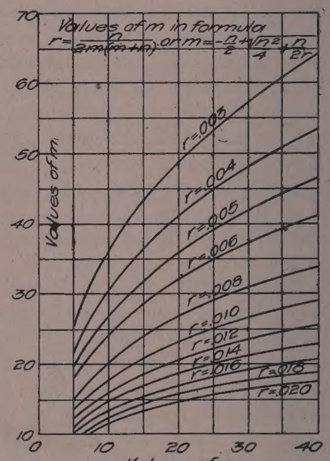
S_c = maximum unit stress in concrete.

S_s = maximum unit stress in steel.

$E_s/E_c = \delta$.

$S_s/S_c = \mu$.

A_s = area of steel in tension.



Values of n

Diagram 6

$$\frac{S_c}{E_c}\Big/\frac{S_s}{E_s} = \frac{yh}{h(1-y)} = \frac{y}{1-y} = \frac{S_cE_s}{S_sE_c} =$$

$$y\left(1 + \frac{S_sE_c}{S_cE_s}\right) \qquad (23)$$

$$y = \frac{1}{1 + \frac{S_sE_c}{S_cE_s}} = \frac{1}{1 + \frac{S_s}{S_cP}} \qquad (24)$$

$$y = \frac{1}{1 + \frac{\mu}{\delta}} \qquad (25)$$

Let A_s/hb equal p. (26)

Then $A_sS_s = 2/3\ S_ceb$ (27)

$$pS_s = 2/3\ S_cy \qquad (28)$$

$$p = 2/3\,S_c/S_sy = 2/3\,y/\mu \qquad 29)$$

Mp, or moment of resistance, =

$$A_sS_sh(1 - 3/8y) \qquad (30)$$

Now, since $y = \frac{1}{1 + \frac{\mu}{\delta}} = \frac{\delta}{\delta + \mu}$

$$Mp = A_s S_s h\left(1 - 3/8 \frac{\delta}{\delta + \mu}\right) \qquad (31)$$

Now for beams of equal strength and equal cost, $Mp = M$; $h = d$; $p = r$; $A_s = S$.

Now in the two methods,

$$M \text{ equals } Mp = QR_sSd$$
$$=.R_sSd\left(1 - \frac{n}{3(m+n)}\right)$$
$$= A_sS_sh\left((1 - 3/8 \frac{\delta}{\delta + \mu}\right) \qquad (32)$$

and if R_s equals S_s,

$$1 - \frac{n}{3(m+n)} = 1 - 3/8 \frac{\delta}{\delta + \mu} \qquad (33)$$

Therefore, $\frac{n}{3(m+n)} = 3/8 \frac{\delta}{\delta + \mu}$,
by assumption δ is constant. (34)

Therefore $u = \frac{n + 9m,}{8}$, or (35)

$$m = \frac{8\mu - n}{9} \qquad (36)$$

If we have a beam to be figured in accordance with the parabolic hypothesis, we have only to take our value for m equal to $\frac{8\mu - n}{9}$ in order to get a corresponding value for Q and M.

If, for illustration $\mu = 40$ and $n = 20$, $m = \frac{320 - 20}{9}$, or 33.33, and $Q = 0.875$.

With the same data and our old formulas,

$$Q = 1 - \frac{1}{3\left(\frac{60}{20}\right)}, \text{ or } 0.889.$$

Since M is proportional to Q, the difference in the resulting strength of beam by the two methods with the same data is about 1½ per cent. for these values of m and n. The difference between μ and m, however, is about 20 per cent. in the above example.

Next considering Capt. Sewell's formula, which is in the same form as equation 18. In his paper read before the American Society of Civil Engineers on Feb. 21, 1906, Capt. Sewell stated that from the result of his studies of the Watertown Arsenal tests the true centroid of the compressive stresses corresponded approximately to an average between the right line and the parabolic figures. If we wish to use this theory with the diagrams accompanying this paper we can do so by making m equal to $\frac{8.5\mu - 0.5n}{9}$ or $\frac{17\mu - n}{18}$

The stress-strain curve here is carried to 80 per cent. of the ultimate strength of the concrete in compression, and the lever arm of the compressive forces is assumed to be fixed for all cases. In other words, a compromise value of Q is assumed for all cases. It is evident from an inspection of Diagram No. 4 that when m is large in comparison with n, Q is proportionately very large. The variation may easily be as high as 10 per cent. This assumption necessarily limits the application of this formula as well as that of the parabolic one.

It seems to be pretty generally admitted that when working stresses are used the right line formulas are correct within the assumptions of the theory. The advocates of the parabolic or other variable stress-strain curve contend that it is bad practice to use anything else than ultimate figures, and that the results should be divided by a factor of safety. When it is considered that the working stresses have been found the best practice on which to go in structural steel specifications, and that the factor of safety is to-day a popular term that means almost nothing mathematically and is grossly misleading anyway, the argument for the use of the right-line formulas gains strength.

From comparisons of a large number of tests, the stress-strain curve for ordinary concrete in compression up to about 800 lb. per square inch departs but little from a straight line. It seems logical that the presence of steel rods at right angles with the line of pressure would have some effect on the stress-strain curve, and it is difficult to imagine any other effect than to straighten it. The writer believes that for some distance beyond the safe load point the right line is the best approximation to the true curve. Also, whatever the hypothesis, a proper choice of m and n, which should always be made by the designer, will enable true values of Q and r to be obtained.

Mr. Leonard Wason's formula assumes that the proper curve is a right line, and furthermore that $x = 1/2$. For this assumption to hold good, m must be equal to n. As has been indicated above, there is a variation of something like 1½ per cent. in the strength of the beam when figured by the rectilinear and the parabolic formulas. It should be noted that this variation, when the rectilinear formula is used, is on the side of safety.

There are a great many formulas besides those given above to the investigation of which the formulas and diagrams of this paper are peculiarly suited. The three quoted have been given by way of illustration. As has been noted at the commencement of this paper, the theory on which these formulas have been derived is not new, and it may be added that a number of those given have been worked out by other methods.

The Erection of the Ferok Bridge.

The erection of the Ferok bridge in India, although done nearly twenty years ago, involved some features that are of interest, both on account of the remote locality where an erection plant was difficult to obtain and because of the modifications adopted at that early date of methods which have since become more common. The bridge comprises six 130-ft. through truss spans supported on cylindrical cast-iron piers 12 ft. in diameter at the base, filled with concrete. It carries a highway and a single track of the Madras Railway across the Ferok River in the western part of southern India, where, according to Indian Engineering, the water is 50 ft. deep and has a very swift current at high tide, thus making falsework erection impracticable. The pier cylinders were made in sections 6 ft. wide, each having segments bolted together with planed inside flange joints calked with rust cement. They were floated into position between bamboo rafts, sunk by dredging and seated on rock by divers. The cylinders diminished to a diameter of 9 ft. above the river bed and were united in pairs by a transverse diaphragm to form each pier.

The trusses were erected in vertical planes in the axis of the bridge on the shore side of one abutment. Near each end of the bridge a wooden gantry was built over it from which the trusses were suspended just clear of the ground. The gantry consisted of two vertical bents parallel to the truss, each made with a center post and two batter posts connected at the top by compound cap timbers transverse to the trusses and seated at the bottom on longitudinal sills under which were placed three rollers made from sections of 5-in. iron piles. The gantries were hauled by windlasses until the truss was pulled across the abutment and to the end of the last span erected, beyond which it projected several panels.

A pair of barges were braced together with cross beams on the decks supporting a 12-ft. pier cylinder. The barges were ballasted with water and at low tide were floated under the projecting end of the truss and tracks being laid to the top of the pier, the gantry was rolled out to bearing over the pier segment, the water ballast being removed to compensate for the additional load on the barges. The barges were towed nearly to the next pier supporting the forward end of the truss, while the rear end was carried by the other gantry. When the latter reached the end of the finished span a second pair of barges with a pier section on them was floated under the trusses as near as possible to the pier and the ballast being removed as before, lifted the truss clear of the gantry and allowed it to be hauled to its final position beyond the end of the completed span, where it was lowered to its permanent seats by a 60-ton hydraulic jack at each end. During the latter operation the rear end of the truss was braced against overturning by inclined struts from the top chord to horizontal transverse timbers secured at the top of the pier cylinder. Each span weighed about 185 tons and was erected by local labor, among which there were many expert bridge men.

Letters to the Editor.

MISTAKES IN WATERPROOFING.

SIR:—An article entitled "Mistakes in Waterproofing," appearing in The Engineering Record of Aug. 18, will doubtless be read with much interest by engineers having to deal with this subject. The problem of waterproofing has in late years become one of the most important which the structural engineer has to solve. Until the last fifteen years but few buildings were erected with a sub-basement, while now it is the general practice in large office buildings, hotels, and other structures, to build at least two stories underground, and three floors below the curb is not uncommon, the natural tendency being to go deeper into the earth as land values increase. The deeper the cellar, of course, the more difficult it is to waterproof.

While the article contains many true and interesting statements, it appears to me that some of them will not stand the light of thorough investigation. Being a fellow member with its writer upon the waterproofing committee of the American Society of Testing Materials, I beg leave to raise a small dissenting voice to some of the statements set forth.

Frankly declaring himself the adversary of all methods which endeavor to check and keep out water with a rigid material, he especially turns his attention to cement waterproofing applied to interior surfaces of walls.

At the very outset the term rigid as applied to any waterproof material is relative—none of those now in common use are as elastic as might be desired,—since it frequently occurs that the best possible waterproofing, employing either felt or paper, though placed with the utmost care, will crack. It is only a question of how great is the settlement or jar sustained by the structure. Cement waterproofing is absolutely rigid and will necessarily crack from a slighter settlement than will felt. Admitted then, for the sake of argument, that both materials will crack. it would seem that the important question is which one can be repaired in the quickest and cheapest manner. As cement waterproofing is applied, wherever possible, upon inner surfaces, it is a matter of so little expense to locate and remedy what few cracks may occur, that no allowance for such repairs is made in the original estimates by the company which waterproofs by the cement method. With the felt system, when the material is placed on outer surfaces, it is frequently impossible to make repairs at all, owing to the interference of adjoining buildings, and in a majority of cases, where the leak may be got at by excavation, a large extent of wall has to be uncovered to locate the source of the trouble, as water, following the line of least resistance, may appear inside the building many feet distant from the point of fracture. The writer of the paper to which I have referred makes little of the advantage of economical repairing enjoyed by cement waterproofing by saying "patching is not perfection" thus dismissing the subject. This would seem a poor stone to fling, when one remembers that with felt and paper waterproofing the entire surface is made up of a series of patches, one overlapping another,—and while "patching is not perfection," with cement it is possible to make a perfect patch. As well object to the use of white plaster in a room on the ground that it cracks with settlement,—the damage in either case being remedied in the same way.

Another advantage of placing waterproofing on inner surfaces is the saving of space. In the heavier buildings, where caisson foundations are used, it is now usual to place a continuous line of them around the entire lot. When the waterproofer steps in his waterproofing must be supported, if felt, or similar materials are used, by a wall hardly less than 8 in. thick, and possibly 20 or 24 in., according to the pressure. With cement waterproofing the material is applied direct to the walls, and is but ⅝ in. thick. The room thus saved will, in a year or two, pay the entire cost of the waterproofing, for I am informed that the rental of floor space in basements and sub-basements in the modern skyscraper of lower Manhattan will average 75 cents per square foot per year.

This statement, "By the very nature of a hard surface, that surface must serve for a while, but also by that very hardness will the waterproof value of the surface be destroyed," is also open to discussion.

Why should the waterproof value of a surface be destroyed by its hardness? Does granite become with time less waterproof than limestone? I claim that as a good cement improves and hardens with age it becomes a more perfect waterproofing agent. As it contains no vegetable or animal matter to decompose, I believe it to be, in fact, the only absolutely permanent waterproofing material now in use.

Another statement, "The hardness of the plaster may keep back water for several years, but it must in time come through and the cement come off,"—may be further discussed. Why must it in time come through? Why must the cement come off? The method of waterproofing to which reference is made is carried on under the guarantee that the cement used "will fulfill its purposes as a waterproofing agent so long as the material on which it is placed remains in a sound and stable condition." This claim can have but one interpretation; a guarantee that contractors will be responsible for the material so long as the structure stands.

We are also told that cement waterproofing cannot be used on a roof, whereas, at this very time the roof of the White House, at Washington, is being so waterproofed.

I agree that experimental briquet, or cement box tests are of but small practical value, but I do hold that where some of the largest cellars in America have been waterproofed with cement, and that tests of this waterproofing after five years' installation show it to be stronger and more impervious to water than when first applied, such tests are of the greatest value, and are conclusive evidence of practicability.

At the end of the article we find: "In applying waterproofing, success depends on the perfection of every detail. The best materials imperfectly applied, are worthless. The smallest hole caused by a dropped axe, crowbar or other tool, unseen and covered, may, after the pumps are stopped ruin the entire work." I heartily agree with the first part of this clause, and maintain that in waterproofing work too much care cannot be exercised, nor will success with any method be secured unless it is carried on under the direction of an intelligent superintendent who has a practical and technical knowledge of the work in hand. But when the writer says that the smallest hole may ruin the entire work he should have made an exception in the case of the new cement system, whereby a hole, large or small, can be stopped with the greatest ease and economy.

Another point in favor of cement waterproofing is that it interferes with the work of other contractors upon the structure less than any other method employed. In fact, the building, especially if it be of reinforced concrete, may be entirely completed, roof and all, before the waterproofers appear on the scene.

In all I have said above I have spoken simply of cement waterproofing, but in order that no one be misled, it is only fair to state that there is but one cement which I know of that is a positive and waterproofing agent. Ordinary cement may be waterproof on small areas, but when applied on large surfaces, hair cracks will develop, and let water through. A hydrolithic cement, however, was invented about six years ago, has been in practical use during that time, and has proven to be a waterproof agent not subject to temperature or shrinkage cracks.

Very truly yours,

New York, Aug. 20. Fred'k S. Greene.

A Clover Covered Private Road.

Sir:—The writer recently saw an interesting piece of road work a few words about which may be interesting to your readers. It was a road connecting a pier on an inland lake in New York State with a palatial summer residence located several hundred feet above the pier on an eminence in a tolerably steep wooden slope, and it had evidently been laid out with special care and under the direction of some one familiar with road building. It takes a zig-zag path to make the ascent from the pier to the house as easy as possible. The grade to all appearances was uniform and quite steep, and the turns of easy curves, these incidentally forming spots that afford resting places for the hauling animal. Old-time summer residents of the locality assert that the road has been built ten or twelve years, and it is this fact that makes it interesting. The entire road bed from top to bottom is covered with a thick growth of short clover grass having every appearance of having been sown for the purpose. The result is an elastic path for the faithful horse, but more important, a protective coating that is proof against the erosion of heavy streams following a rainfall. The spectator at the top has a strong temptation to take a slide or roll down the turf, while a horse on the upward climb is pretty sure to sample the clover at his resting stops. At any rate, as an indication of the possibilities of reducing repair costs for a roadway subjected to little traffic but to many torrents, it is probably worth mention, especially when so many roads on no greater and even less grades in the vicinity are badly rutted and washed by storm water.

Highway Engineer.

Roads and Tires.

The editorial comment on Roads and Tires in The Engineering Record for July 28, last, is certainly on a timely subject and one that is of partircular interest to all who are concerned in the construction and maintenance of city roads.

Our crying need here, in Chicago, is some kind of road surface that will withstand the heavy automobile traffic to which our boulevards and park drives are subjected. On some of the boulevards there are over 5,000 vehicles passing per day, of which about 75 per cent. are automobiles. The most desirable road surface for automobile traffic, as well as driving is a smooth, well-kept macadam roadway. The action of the rubber tires of the automobiles seems to be to pick up the loose stone of the surface by suction, in this manner starting holes or pits in the road surface, which gradually enlarge and increase in number until it becomes necessary to regrade and reroll the drive. The more fine material, or binder, there is used in the macadam the more this picking up action is noticed, consequently we get a more durable road surface by using the smallest possible quantity of binder. It does not seem to matter very much whether the binder used is granite or limestone; neither one stands well under automobile traffic.

We are now doing some experimental work with tar surfaced roadways. The tar we are using is of a moderately heavy grade, poured on hot just after the macadam surface has been well rolled. In some cases we have applied two coats of tar to the surfaces and in others only one. After the tar has been poured on and allowed to penetrate the rolled surface of the macadam as much as it will, we spread on a thin layer of limestone screenings to absorb the surplus tar, compacting it with a final light rolling. The object is not to make the tar penetrate the macadam to any great depth, only far enough to form a sort of cushion and to prevent the picking up of the stones by the rubber tires.

We are also trying, in a smaller way, a concrete surfaced roadway by pouring on the coarse stone, after being rolled, a grout composed of cement one part and fine limestone screenings or sand two parts. This will hardly give an ideal surface for driving, but it appears that it might be successful for automobile travel. After the grout is poured on and before it has taken its initial set the surface is rolled with a light roller and spread over with a thin coating of torpedo sand. Enough grout is used to fill the upper 2 to 4 in. of coarse stone, which is first rolled to grade without any binder. We used what is called 1½-in. limestone which, when rolled, had about 25 per cent. of voids. This gave a basis upon which to calculate the quantity of grout required. No provision has been made for expansion joints in this concrete surfaced roadway, nor was it laid in blocks, as is customary in smooth finished concrete pavement.

Neither the tar nor the concrete surfaced roadways have been in use long enough for me to give any data as to results, nor have I the costs of either at hand in a tabulated form.

Yours very truly, Linn White, Engr.
South Park Commissioners.
Chicago, Ill., Aug. 21, 1906.